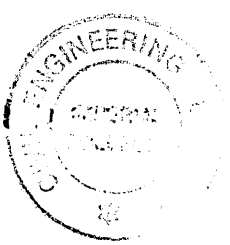
Shearing Resistance of Natural Quick Clays.

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Abstract.

The purpose of the study described in this thesis was to investigate Norwegian quick clays. The study was divided into field investigations of in situ stress measurement (K_o) and of failures of natural slopes, and laboratory studies of physico-chemical characteristics of minerals and soils, and of the behaviour of quick clays for different types of failure processes.

It was concluded that normally consolidated quick clays have anisotropic undrained strengths, that in overconsolidated states they do not have the pronounced strain-sensitive characteristics, whereas when they exist in the normally-consolidated state they can be very strain sensitive and can be involved in progressive type failures.

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Chapter 1. Subject Review and Methods of Investigation.

1. Introduction.

The title of this thesis requires some explanatory remarks. First, the term quick clay refers to those clays that in a remoulded state have the consistency of a thick fluid and an undrained shear strength less than 0.02 t/m^2. This definition is in keeping with the original meaning of the Scandinavian word "kvikk" used to describe the clay that flowed away from the sites of certain landslides and that often caused much trouble by "flowing" into shallow excavations during excavation. It is these soils, that can become fluid, that will be investigated in this thesis. The more common idea of a quick clay is one having a sensitivity greater than about 30 , and such a definition would include most "kvikk" clays, but this definition is unacceptable for this thesis because it also includes certain very stiff clays that when remoulded become very soft compared to their undisturbed state, but yet these remoulded clays are far from being fluid. Second, it might be thought from the thesis title that quick clay is a species of clay. In fact quick is a group name that includes clays of different geologic history and geographical location but having the common characteristic of small remoulded strength. In this thesis emphasis will be placed on Norwegian marine quick clays, and therefore most of the conclusions drawn will apply directly to Norwegian clays, and will not necessarily be applicable to Canadian and Swedish fresh-water quick clays. Third, the term shear resistance was specifically chosen in preference to the often-used phrase shear-strength properties, because of two reasons: (i) shear strength is generally associated with macro-failure, whereas shear resistance is a broader term not necessarily related to failure, and (ii) shear strength is frequently not a property but rather a behaviour dependent on properties of the mineral particles but also dependent on milieu (particle arrangement, stress, drainage, etc.).

In the following sections of this chapter a brief review will be made of developments of knowledge concerning sensitive and quick clays up until the time this author began his work in this subject. Finally, an outline will be given of the program of investigation that was followed and that provided the information to be presented in the following chapters.

2. Developments up to 1959.

By the year 1959 (the time of the Boulder Shear Strength Conference and also the year of the Furre landslide that led to extensive field and laboratory investigations in Norway) a great amount of engineering experience had been gained with soft sensitive clays. Swedish work was outstanding and provided many of the foundation stones for engineering design and practice in soft clays (Bjerrum and Flodin, 1960; and the publications of the Royal Swedish Geotechnical Institute, Stockholm). Methods were developed for obtaining undisturbed samples, for measuring what today is called undrained shear strength but earlier was simply called shear strength, and for calculating stability. In most cases that were reported, it was found that the strengths obtained from such tests as the fall-cone, vane and unconfined compression compared very well with observations in the field of bearing capacity, bottom heave, and slope failures, and it was therefore thought that the measured strengths gave a true picture of the soils' strength properties.

Although the strength properties of sensitive clays had been studied successfully, investigations of quick clays had been largely unsuccessful, presumably because these clays presented great difficulties in sampling and testing. However, much attention had always been given to quick clays because of the devastating landslides that could occur in them. It is perhaps not surprising that there has never been a lack of hypotheses to explain the remarkable landslide behaviour of quick clay, these hypotheses being as varied among the common folk as among men in science. Among the latter group, for example, it has been thought that quick clay existed as a fluid trapped beneath a surface layer of stiff clay, and if for some reason the stiff clay became ruptured, the quick clay would flow out (Reusch, 1901);

it has also been thought that quick-clay slides were preceded by extensive internal erosion and that finally the eroded tunnel collapsed, the falling clay liquified and caused further erosion until the main slide occurred (Terzaghi and Peck, 1948); and it has been thought that quick clays are similar in behaviour to very loose sand, and that small disturbances cause the whole soil fabric to suddenly collapse to become a fluid. The cause of the high sensitivity and low remoulded strength of Norwegian quick clays had been shown to be the leaching out of the salty pore fluid (Rosenqvist, 1955). The fact that leaching also decreased the undrained shear strength of the undisturbed clay (Bjerrum and Rosenqvist, 1956) supported the general feeling that marine clays in nature that were subjected to fresh-water seepage became weaker with time and it was one of the causes of landslides in quick clays.

Thus, during the years preceding about 1953, most people thought in terms of undrained shear strength (total stresses). Although this line of thought had proved successful in many ways, there were weaknesses, and the most important was the use of undrained strength to estimate the stability of natural slopes. Here, the method had proved unsuccessful in predicting failure of several natural slopes and it had given safety factors less than unity for obviously stable slopes (Bjerrum and Kjærnsli, 1957).

By 1959, following the Conference on Measurement of Shear Strength of Soils (London, 1950), the Conference on Stability of Earth Slopes (Stockholm, 1954) and the International Soil Mechanics Conference in London (1957), the use of effective stresses to interpret strength tests and to calculate stability had gained general acceptance in principle, due mainly to the outstanding work of J. Hvorslev, D.W. Taylor, A. Casagrande, A.W. Skempton, A.W. Bishop and L. Bjerrum. In practice, however, the stability of soft clays had seldom been calculated by this method and in a review of the situation, Bishop and Bjerrum (1960) could only list two Norwegian slope failures that had been investigated in terms of effective stresses, and neither of these clays were quick. It is of interest that for both cases the c'φ' analyses apparently accurately predicted unstable conditions, whereas the use of undrained shear strength in the s_u analysis resulted in safety factors equal to

and appreciably less than unity. Strength behaviours in terms of effective stresses of sensitive and quick clays had been extensively investigated in the laboratory, and the consensus on these results was that the measured behaviours of soils in the laboratory were not similar to those in situ. The reasons given for rejecting the test results were usually that the laboratory c' φ' parameters were too large and that they failed completely to predict the small undrained shear strength measured in the field, and also that the usual laboratory test procedures caused large reductions of volume and sensitivity of the test specimens, and these reductions were incompatible with field evidence. Explanations that were given for these laboratory failings usually involved sample disturbance (Bjerrum and Simons, 1960), incorrect stress and strain conditions and the neglect of such factors as rate of strain (Crawford, 1960), and faulty test interpretation (Kenney, 1960). Thus, in 1959 there was little agreement concerning the magnitude or the method for measurement of the effective-stress shear-strength parameters of natural sensitive clays, and there was essentially no favourable experience with the prediction of stability of sensitive clays through the use of effective-stress methods.

3. Development in Norway, 1959-1962.

In 1959 a large landslide occurred at Furre, Norway, where a huge flake of sand slid out along a thin inclined layer of quick clay. This slide was thoroughly investigated (Hutchinson, 1961) and the results of stability calculations indicated that the effective-stress shear-strength parameters of the quick clay were $c_d \approx 0$, $\text{tg}\,\varphi_d \approx 0.12$. This result was checked and apparently confirmed by in situ drained shear-box tests. Concurrently, much work was being done to study the shear strength of very loose sand and the results of tests indicated that such sands in drained compression could exhibit such low values as $\text{tg}\,\varphi_d \approx 0.36$ (Bjerrum, Kringstad and Kummeneje, 1961). These results for clay and sand were considerably smaller than any results previously obtained, and by using $\text{tg}\,\varphi' \approx 0.12$ for quick clay, correct predictions could be made of the small undrained shear strengths of quick clays

measured in the field (Bjerrum, 1961).

To supplement these findings the Norwegian Geotechnical Institute started a detailed program of laboratory testing of a typical quick clay from Oslo (Landva, 1962; Bjerrum and Landva, 1966). The program first concerned the development of a number of special apparatus for shear testing of quick clays; a direct simple-shear device, apparatus for performing vane tests in triaxial cells, and apparatus for minimizing disturbance during sample preparation for all types of tests. The testing program consisted primarily of tests on samples reconsolidated to approximately the vertical effective stress acting in situ for the condition during consolidation of zero radial strain (K_o condition). A brief summary of the results of the undrained (or constant-volume) tests is given below, where s_u denotes the measured undrained shear strength (that is $\frac{1}{2}(\sigma_1-\sigma_3)$ for compression tests, τ_{max} on horizontal planes in the direct simple-shear tests, and τ_{max} in the vane tests) and P' denotes the major principal effective stress acting during consolidation.

Results of Undrained Tests on Manglerud I Clay.

Field vane and unconfined compression tests	$s_u/P' \approx 0.13,\ 0.17$
Laboratory K_o-triaxial vane tests	$s_u/P' \approx 0.19$
Laboratory direct simple-shear tests	$s_u/P' \approx 0.19$
Laboratory K_o-triaxial compression tests	$s_u/P' \approx 0.30$

These test results were of considerable importance because, for the first time, laboratory tests on reconsolidated samples of quick clay gave results of undrained shear strength similar to those measured in the field. The laboratory K_o-triaxial compression tests remained an exception, corresponding to all previous experience that these tests gave overestimates of s_u, in spite of the fact that Landva had made great advances in reducing sample disturbance in the laboratory and also in spite of the fact that disturbances to triaxial-cell samples would have affected both the vane and the compression tests, whereas it appeared to have affected only the compression tests.

When interpreted with respect to effective stresses, these tests did not give the expected small values of tg φ', similar to those obtained for the Furre clay. It was found that in the direct simple-shear tests the shear resistance was increasing when the tests were stopped, at which time tg $\varphi' > 0.3$. The K_o-triaxial compression tests gave tg $\varphi' \approx 0.5$, as compared to a mobilized tg φ' at the end of K_o consolidation equal to tg $\varphi'_{K_o} \approx 0.35$.

This was the state of knowledge concerning the behaviour of quick clay when the author of this thesis began work on the subject in 1962. There appeared to be a large number of consistent facts, but there also existed some important inconsistencies. First, it was known from field experience in Norway with excavations and loadings that quick clay could be very weak, an observation well supported by the slide at Furre and the field-measured tg $\varphi_d \approx 0.12$, but if the drained shear resistance of quick clay was commonly this small, there would exist only very gentle slopes in quick clay and this was not consistent with the results of field observations. Second, the laboratory test data gave a very inconsistent picture; the undrained tests indicated that the quick clays were very weak, whereas with respect to effective stresses the shear strength parameters indicated strong soils. Third, the results of the K_o-triaxial consolidation tests indicated that (i) the shear stresses during consolidation were larger than those measured at failure along horizontal planes in direct simple-shear tests and were also larger than those measured in vane tests after K_o consolidation, and (ii) the value of mobilized tg φ during K_o consolidation of a quick clay was very much larger than the drained failure tg φ reported for the Furre clay.

4. Methods of Investigation.

The plan that evolved for continuing the investigation of shear resistance of natural quick clays was essentially to study separately the points of inconsistency listed above.

1. K_o-consolidation. A sample existing at K_o conditions (consolidated for zero radial strain) is more stable than a sample experiencing shear deformations after K_o consolidation. The idea was that natural quick clays have experienced K_o consolidation and therefore the mobilized shear resistance during consolidation could be used as a minimum measure of the shear resistance of the clay. Hence, $\text{tg}\,\varphi_{K_o}$ mobilized during K_o consolidation should be less than $\text{tg}\,\varphi'$ at shear failure, and also the maximum shear stress during consolidation should be less than the shear strength of the soil after K_o consolidation. If K_o was found to be large and approaching unity, this would indicate the possibility that $\text{tg}\,\varphi'$ and s_u/P' were small, but if K_o was small this would indicate that both $\text{tg}\,\varphi'$ and s_u/P' were large.

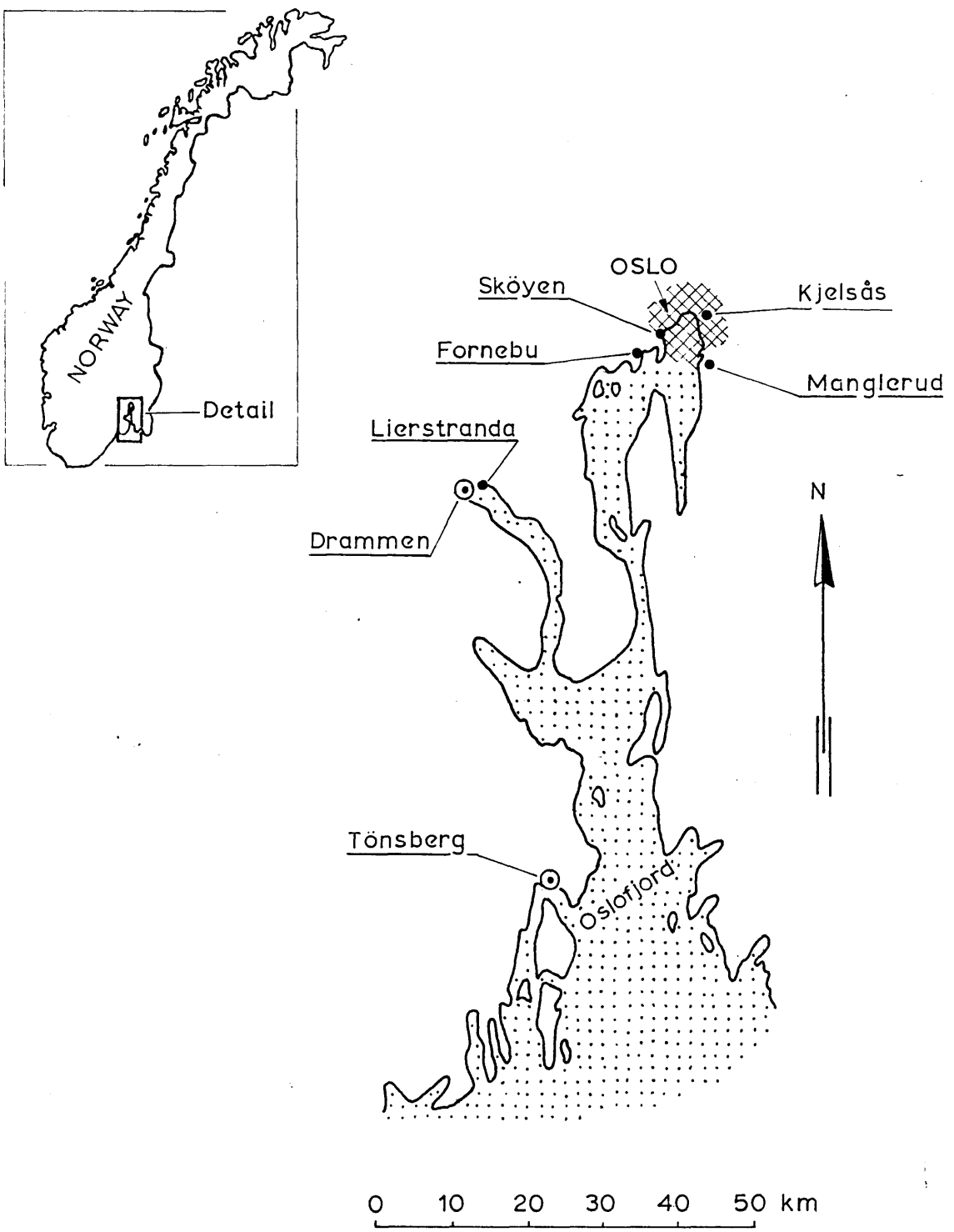
2. Slope failures. The shear resistance of quick clay could be directly studied by investigating slope failures in quick clay, and by considering these failures as large field tests. Time was not available to extend this study to include stable, natural slopes.

3. Strength properties of minerals. In order for clays and sands to exhibit small drained shear resistances, it is necessary that one of the constituent minerals, or the mineral mixture, or either of these in combination with the pore fluid, should exhibit small drained shear strength. Therefore, tests were performed to determine the strength behaviour of minerals and mineral mixtures in drained shear tests and in consolidation tests, and also to determine the influence of certain natural system changes, such as a reduction of ion concentration in the pore fluid and the replacement of exchangeable cations.

4. Interpretation of tests. Considerable differences of behaviour were measured for supposedly identical samples tested in different apparatus. If the samples were in fact identical, their properties would also have been identical, and the measured behavioural differences would be due to differences of conditions of testing. Therefore, the various methods of testing were studied and compared to find any major differences between them and to determine how these differences would affect soil behaviour. Sample disturbance was also considered.

These four points will be dealt with in the following four chapters.

Sites from which samples were obtained for testing and at which field tests were made are located in Fig. 1-1.



SITES OF SOIL SAMPLING AND FIELD TESTING

Chapter 2. Measurement of K_o for Sensitive and Quick Clays, in the Laboratory and in the Field.

1. Introduction.

K_o is defined as the ratio of the radial to axial principal stresses for the condition of zero radial strain in a soil element; that is, $K_o = \sigma_r' / \sigma_a'$. This condition corresponds to the compression of a horizontal semi-infinite soil mass when loaded vertically over the whole horizontal surface, and in nature corresponds to the environment of clay elements during deposition and beneath extensive terraces after deposition. The K_o condition is also the stress condition of maximum stability for a normally-consolidated soil[x] and in this condition the maximum shear stresses acting within an element should be a minimum measure of the maximum effective-stress shear strength parameter: that is

$$\frac{\frac{1}{2}(\sigma_a - \sigma_r)}{\frac{1}{2}(\sigma_a' + \sigma_r')} = \frac{1 - K_o}{1 + K_o} = \sin \varphi_{K_o} < \sin \varphi'_{max} \quad \ldots\ldots\ldots \quad 2.1$$

where φ_{K_o} is the mobilized angle of shearing resistance for K_o conditions and φ'_{max} is the maximum angle of shearing resistance for macro-shear failure conditions. Similarly, the ratio of the maximum shear stress to the maximum axial effective stress existing during consolidation should also be a minimum measure of the ratio of maximum shear strength (drained or undrained) to consolidation pressure; that is,

$$\frac{\frac{1}{2}(\sigma_a - \sigma_r)}{\sigma_a'} = \frac{1}{2}(1 - K_o) = \left(\frac{\tau_{max}}{p'} \right)_{K_o} < \frac{s_{max}}{p'} \quad \ldots\ldots\ldots \quad 2.2$$

The above statements are correct but the problem is not as clear and as simple as might be implied by equations 2.1 and 2.2. First, in equation 2.1 the term angle of shearing resistance φ' has been used in the

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x) Strictly speaking, more stable conditions could be obtained by increasing the radial stress after which the element would still be in a normally-consolidated state.

most general sense to include all shear strength components, be they friction, cohesion and/or dilatancy. Thus the term φ' does not denote only a frictional property. Second, in equation 2.2 comparison has been made between maximum shear resistance mobilized during K_o consolidation (τ_{max, K_o}) and maximum shear strength developed at macro-shear failure (s_{max}). It will be discussed in detail in following sections of the thesis that mobilized shear resistance and shear strength are not soil properties but are soil behaviours, the difference being that a behaviour is dependent not only on soil properties but also boundary stress, strain and drainage conditions. Hence, if on a system of parallel planes through an element, maximum shear strength s_{max} is strongly dependent on cementation bonds between particles and these bonds are easily broken, it follows that if failure occurs on these planes the cementation-bond component throughout the element will decrease and hence the maximum shear strengths that can be later developed on any other planes will be less. This point can also be shown by considering the effect of strain on pore pressure and the pore-pressure effect on strength on planes of different inclination. Therefore, although equation 2.2 is correct in stating that:

$$\left(\frac{\tau_{max}}{p'}\right)_{K_o} < \frac{s_{u_{max}}}{p'}$$

it does not follow that $(\tau_{max})_{K_o} < s$ on any plane within the element.

The main purpose of the K_o investigation was to attempt to determine whether K_o for quick clays was large or small. If K_o was found to be large (small shear stresses within the soil) it would indicate that the small value of tg φ' reported for Furre clay could be correct. A large K_o-value would also be consistent with small measured values of s_u/P'. On the other hand, if K_o was found to be small, similar in magnitude to $K_o \approx 0.5$ measured by Landva (1962), it would indicate that tg $\varphi_{K_o} \approx 0.35$ and $(\tau_{max}/P')_{K_o} \approx 0.25$, values appreciably larger than those thought to be correct for quick clay, and therefore it would indicate that the problem was rather more complex than generally thought and that it probably involved measured behavioural phenomena rather than property phenomena.

The K_o investigation involved three types of measurements; laboratory K_o-triaxial consolidation tests, laboratory K_o-consolidation tests with rubber membranes reinforced with a spiral winding of electric-resistance wire, and field K_o measurements with a pipe instrumented with gauges that was installed in the ground at three sites.

2. K_o-Measurement with Standard Triaxial Apparatus.

The test method that was used was one reported by Bishop (1958), where a sample was stressed by increasing the cell pressure (radial stress) in increments and increasing the vertical pressure (axial stress) for each increment such that the average cross-sectional area of the sample remained constant and therefore radial strain was zero. This meant that the ratio of the volume of expelled water to the change in sample height remained constant; that is, $\Delta V/\Delta h$ = Area = constant. This test has obvious failings; increases of radial stress will primarily cause decreases of sample area, whereas increases of axial stress will primarily cause increases of sample area, and if the radial stress is increased too rapidly the area will decrease, whereas if the axial stress is increased too rapidly the area will increase, both these errors violating the zero-radial-strain condition for K_o testing. In practice these errors are minimized by using small stress increments. For his work, Landva had used from 2 - 4 radial-stress increments of $\Delta\sigma_r \approx 0.2$ kg/cm^2 to reach the desired level of consolidation stress. As a possible improvement over this method, it was decided to use smaller stress increments beginning at $\Delta\sigma_r = 0.05$ kg/cm^2, but later these were reduced to $\Delta\sigma_r \approx 0.025$ kg/cm^2. This value is very small, but also the overburden stresses acting in situ on the samples that were tested were small, being $P_o' \approx 0.5$ kg/cm^2, and most of the K_o testing work was done at stresses approximating P_o'. For those tests in which $\sigma_a' >> P_o'$, the $\Delta\sigma_r$ increments were increased as the test progressed.

All the soils that were tested were undisturbed clays obtained from the field with 9.5 cm diameter piston-sampler equipment. The samples were 1 metre in length and as standard practice 10-15 cm of soil at each end of the sample was discarded to avoid possibly disturbed soil. An extremely

critical attitude was taken towards sample acceptance and much material was discarded because of obvious or suspected disturbance, determined by frequent measurements of strength and sensitivity with drop-cone tests. Specimens 8 cm high and 10 cm^2 in sectional area, were prepared and installed in the triaxial cells by means of special apparatus (Landva, 1964), without which undisturbed samples could never have been obtained. The undrained strengths of most of the materials that were tested were less than 1.5 t/m^2, with sensitivities frequently greater than 100, and these properties left very little margin for error in laboratory work. Each sample was surrounded by slotted paper drains that covered 25% of the peripheral area, and by two thin rubber condoms. Because small cell pressures were used and because the rubber membranes were of excellent quality, no significant membrane leakage was obtained even though some tests lasted longer than 40 days (see also Olson, 1964; Poulos, 1964).

During the first tests, in which the first cell pressure increment was 0.1 to 0.2 kg/cm^2, it was found that relatively large volume changes occurred ($\Delta V/V > 1\%$) and that in attempting to keep the ratio $\Delta V/\Delta h$ equal to a constant, large axial stresses had to be applied and as a result some of the samples failed. It was soon realized that the large volume decreases at pressures equal to a small fraction of the overburden pressure could not be a reflection of the properties of undisturbed clay but rather must have been the result of disturbance. The samples were checked and were found not to be measurably disturbed, leaving only the possibility that a thin zone surrounding each sample had been disturbed due to the cutting operations involved in sample preparation. Such a thin zone of disturbed quick clay could easily provide 1% volume change at small cell pressures, as shown by the typical compressibility characteristics of quick clay given in Fig. 2-1, and the expulsion of this water at small cell pressures would not materially affect the volume of the inner body of undisturbed clay. However, in attempting to keep the sectional area of the sample equal to the initial area, the inner body of undisturbed clay had to experience radial strain (see upper diagram in Fig. 2-1) and as mentioned above these radial strains were sometimes large enough to induce shear failure of the sample.

Following this experience the standard test procedure that was adopted was as follows. The sample was consolidated under a series of cell pressures, usually beginning at $\sigma_r = 0.05$ kg/cm^2 and increasing in increments of 0.025 to 0.05 kg/cm^2 up to a cell pressure of from 0.10 to 0.20 kg/cm^2. Volume-change and height-change measurements were made and this procedure was stopped when the volume changes became small, most of the water being expelled during the first few applications of cell pressure. New dimensions of the sample were determined; $h_i = h_o - \Delta h_c$, $A_i = (V_o - \Delta V_c)/h_o - \Delta h_c)$. The cell pressure was then increased in increments of 0.025 to 0.05 kg/cm^2 and the sectional area was maintained equal to $A_i (1 \pm 0.001)$, this corresponding to a radial strain of $\pm 0.5 \times 10^{-3}$. To simplify the test, use was made of the following relationship.

$$\begin{aligned} \Delta V &= A_i h_i - Ah \\ &= A_i h_i - A_i (1 \pm 0.001)(h_i - \Delta h) \\ &\approx \pm 0.001 A_i h_i + A_i \Delta h \end{aligned} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 2.3$$

This relationship can be drawn as three parallel inclined lines in a ΔV, Δh diagram, two lines giving the allowed deviation limits and the centre line describing the condition of $A = A_i$. If the measured values of Δh and ΔV plotted off the centre line, it meant that either the cell pressure or the axial load had to be increased. This procedure was easily followed but the work involved was tedious and the tests frequently lasted more than one month.

Typical results are given in Fig. 2-2 and information concerning these tests and the soils tested is given in Table 2-1. Each set of information in Fig. 2-2 is given by two diagrams, the upper diagram being a plot of the principal stresses σ_a' and σ_r' and the lower diagram being a plot of axial stress and axial strain (also volume strain). The first figure is a typical test result, chosen to indicate the manner in which the stresses and strain changed. It was found typical that at stresses less than ca. $1.5 P_o'$, the ratio $\Delta\sigma_r'/\Delta\sigma_a'$ was considerably larger than at stresses greater than ca. $1.5 P_o'$. The second figure indicates the effect on the ratio of the principal stresses, $\sigma_r'/\sigma_a' = K$, by changing the sectional area by $\pm 0.1\%$. It is shown that at stresses considerably larger than P_o', area variations have little effect on

K for this soft clay. The effect at stresses less than P_o' has not been measured but it could be anticipated to be larger.

The third and fourth figures present results typical of tests performed on Manglerud III and Drammen clays. In all tests K_o decreased with increasing stresses to values equal to or less than 0.5, and for larger stresses remained equal to $K_o \approx 0.5$, independent of volume changes. In looking at these drawings it can be seen that the K_o - portions of the tests were started when the samples were experiencing large σ_r'/P_o' isotropic stress, and that the test method could have been improved by first decreasing the cell pressure to about zero after the disturbed soil was initially consolidated, and from this stress condition beginning the K_o test. This would have altered considerably the resulting stress paths in the small-stress range but probably would not have altered the results in the large-stress range.

The most important result of these tests was that reconsolidated Manglerud III clay, with St ≈ 50, gave the result $K_o < 0.6$, a result essentially equal to that found by Landva using much cruder testing methods. This result was also obtained for reconsolidated insensitive Drammen clay (St ≈ 10), in spite of the fact that the soils were quite different with respect to plasticity and behaved rather differently with respect to compressibility.

Although the tests on quick clay were consistent, there were unanswered questions concerning effects on the behaviour of quick clay of field sampling and volume decrease during reconsolidation in the laboratory. Could such factors as these affect the clay samples in such a way that their in situ behaviour could not be reproduced in the laboratory? Rather than first trying to answer this question by attempting to obtain even better samples from the field and performing the tests even more accurately, it was decided to obtain samples of a natural insensitive salt clay that was normally consolidated, and to convert this clay to a more sensitive clay, hopefully a quick clay, by leaching it with fresh water (Bjerrum and Rosenqvist, 1956). This approach would circumvent most of the uncertainties concerning sample disturbance during field sampling and laboratory handling and reconsolidation. To pursue the program it was necessary to use apparatus with which the sectional area could be maintained constant, the radial stress

could be determined, and the sample could be leached by a flow of water. Although the triaxial apparatus could have been used for this purpose, the test accuracies would have been very poor if the previously described triaxial procedure had been followed, simply because the volume-changes could not have been measured accurately enough. Therefore, a new apparatus was developed, and it will be described, along with the test results, in the following section.

3. K_o-Measurement with Reinforced Rubber Membranes.

Apparatus.

For use with the NGI direct simple-shear apparatus there had been developed a rubber membrane reinforced with a spiral winding of fine suture wire (Landva, 1962). The idea that was pursued by the author was to replace the suture wire by electric-resistance wire and to convert part of this spiral winding into an electric-resistance strain gauge that could be used, by direct calibration, to measure radial stresses.

A brief description will first be given of the test arrangement; it was similar to that shown in Fig.2.4. The soil sample, enclosed within the reinforced rubber membrane and two end pieces containing porous plates, was placed in a consolidation press having a guided loading plate; this was necessary because the membrane had no resistance to lateral deformation. The consolidation press was located on a table that stood 1 metre above floor level. The end pieces had pairs of tubes leading to the porous plates. A flask containing the fluid that was to be permeated through the sample was connected to a tube leading to the upper end piece and the fluid level in the flask was kept between 1 and 2 cm above the clay sample. A tube from the lower end piece was allowed to hang to the floor and when filled with water provided a seepage head of 1.2 metres. Seepage water was collected in a flask. The thicknesses of all samples were about 20 mm. and the sectional area was 50 cm^2.

The most difficult task was to construct a suitable reinforced membrane. The requirements for the membrane and its associated strain-

measuring equipment were (i) that the membranes allow only small radial strains, the tentative maximum value being set at about 1×10^{-3} (comparable to the apparent accuracy of the triaxial-cell K_o tests where the allowable radial strain was set at $\pm 0.5 \times 10^{-3}$), (ii) that the membranes be capable of reacting to changes of stress less than 0.01 kg/cm^2, and (iii) that the whole measuring apparatus remain stable (no change of zero reading and no change of membrane calibration) over time intervals of months while the membrane was in contact with clay saturated with pore fluid that could vary from salt water to fresh water.

The final product is shown in Fig. 2-3. The membrane consisted of an inner impervious rubber layer consisting of two thin layers of prevulcanized latex and a middle layer of vulcanized butyl latex, that is very impervious. Over this inner layer was wound a pair of constantan electric-resistance wires 0.15 mm diameter, and to obtain an electric strain gauge over the middle 10-mm height of the winding a bifilar winding was made by joining with solder the two wires at one end of the gauge section and by fastening copper leads to each of the two wires at the other end of the gauge section. These operations were done before the wire was wound over the rubber in order that the soldered junctions could be carefully painted with insulation lacquer and that the insulation on all the wire could be accurately checked for weaknesses and repaired if necessary. The outer rubber layer of prevulcanized latex was then built up over the windings. The reason for using different types of rubber was to eliminate the serious problem of short circuiting due to water entering the rubber by diffusion. (In all tests the water pressure in the clay was zero, or negative due to suction, except for short periods of time following load increases). The problem of water diffusion could not be solved by improving to the greatest of our ability the insulation lacquer on the wires. Therefore, the inner rubber layer was made as impervious as possible (0.8×10^{-12} cm^4/gm.sec) to prevent as much as possible the movement of water from the soil into the rubber, whereas the outer layer was made as pervious as possible ($> 30 \times 10^{-12}$ cm^4/gm.sec., or greater than 40 times as pervious as the inner layer) so that, in combination with a drying agent (silicagel) placed outside around the membrane and that maintained a relative humidity of about 9%, any water passing through the inner layer would be rapidly withdrawn from the rubber

in contact with the wires without being able to have any short-circuiting effect.

The measuring system consisted of a standard electric strain indicator (used commonly with commercial electric-resistance strain gauges) that formed a bridge when connected with the active membrane containing the soil sample and a passive or dummy membrane (Øien, 1962). Connected in parallel with the active and passive membranes were condensers. The variable condenser in parallel with the dummy membrane was used to balance out inequalities of capacitance that would otherwise have caused electric resistance due to impedance and not due to strain in the membrane wire, and also to measure changes of capacitance during the test. Water diffusing rapidly from the sample into the membrane would cause a sharp increase of the capacitance of the active membrane, indicating the possibility for short-circuiting, and therefore any membrane that exhibited marked increases of capacitance shortly after the sample was installed was rejected and the tests were stopped. The measuring system was sensitive to temperature changes and therefore all tests were performed in a room having a temperature of $21.5 \pm 0.3^{o}C$.

Although the membrane acted as a strain gauge (the strain indicator registered directly in micro-strain units), radial stresses were determined from pressure calibrations of the membranes, account necessarily being taken of narrowing of the gauge section due to axial strain ϵ_a of the sample by using the equation:

$$\sigma_r = \sigma_{r\,calib.} / (1-\epsilon_a)$$

The calibration apparatus (Fig. 2-3) consisted of a cylinder having a thin rubber sleeve, against which known water pressures could be applied. The membranes to be calibrated had essentially no strength in their axial direction (only the strength of the rubber) and at small pressures they would fail locally, a bubble forming between two wires at the weakest point in the rubber. To prevent the membrane from deforming axially in this manner, a band of abrasion cloth, slotted vertically so as not to hinder the deformation of the inner rubber sleeve, was placed between the sleeve and the membrane and the abrasion cloth prevented the membrane from locally deforming in the

axial direction. Before each calibration test the membrane to be calibrated was loaded to the maximum operating pressure (1.4 kg/cm^2) and unloaded at least 20 times in order to minimize hysteresis behaviour. The membrane was then removed from the calibration apparatus by means of a vacuum cylinder and transferred to the sample to be tested. The membrane was not removed from the vacuum cylinder between the time of calibration and the time of sample preparation because handling it would have caused disturbance of the windings and changes of membrane temperature and thereby changes of the zero point.

The general method of preparing a sample has been described by Landva (1962) (Fig. 2-4). It consisted of using a cutting ring guided on two vertical rods on which the vacuum cylinder for the rubber membrane also could be mounted. The sample was first cut to the correct size by means of the cutting ring, the ring was then drawn up, and finally the membrane was drawn up over the sample. During all operations membrane-strain measurements were observed to check if the membrane was being altered.

The sample preparation apparatus and the membrane vacuum cylinder were built with small tolerances ($\pm$ 0.015 mm). The membranes could not be made with similar tolerances. Therefore there is little doubt that slight surface disturbance of the sample occurred during mounting of the membrane, and this disturbance could have been additional to some surface disturbance that no doubt was caused by the cutting ring. Therefore, the net result was a sample similar to that in the upper diagram of Fig. 2-1.

A typical test result is given in Fig. 2-5. It should be noticed that the apparatus remained stable under variable stress conditions over a long period of time during which the pore fluid in the clay sample was changed from salt water (excellent short-circuiting properties) to essentially fresh water (poor short-circuiting properties). The effect of changes of electrical conductivity of the pore fluid on the behaviour of the membrane was investigated very thoroughly, and the results of typical tests are given for the time period Day No. 147 to 199. It is shown that changes of fluid in contact with the unstressed membrane had no measurable effect. With this in mind, it is of interest to note the membrane strain measurements after Day No. 48 when leaching was begun. The strain measurements increased, indicating larger

measured electric resistances, and these could have been due to short-circuiting caused by the salt water, except that in the later membrane tests the membrane had been found to be stable for changes from fresh to salt water. Concerning the sensitivity of the membrane to register changes of radial stress, it will be noticed that close to Day No. 70 two small changes were made of suction applied at the base of the sample (0.04 and 0.02 kg/cm^2) and easily-measured changes of membrane strain were obtained.

One characteristic of the membrane that has not been mentioned is its resistance to axial compression; that is, the amount by which it reduced the axial load on the soil. No method was found whereby this resistance could be directly measured, and the corrections that have been made were based on calculations in which the elastic modulus of rubber was chosen to be 18 kg/cm^2. For the soils dealt with in this thesis the correction to the applied axial stress was never larger than -5%.

Results of tests on Sköyen clay.

Sköyen is located at sea level on the edge of Oslofjord. The clay at this site is post-glacial in age, extends to depths of 25 metres below sea level and is a normally-consolidated and unleached clay with a pore fluid containing 27 to 32 gm equiv. NaCl/litre[x]. Some of the properties of the clay are given in Table 2-2 and summarized below.

$w \approx 40\%$ $\quad w_L \approx 52$ $\quad w_P \approx 24$

$I_L \approx 0.5$ $\quad St \approx 5$

The soil consists of 50-60% by weight quartz and feldspar and the remaining minerals are hydrous mica and chlorite divided as a 2:1 ratio. The soil profile at Sköyen has been reported under the name Skabo by Kenney (1964)

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x) In addition to sodium chloride there are also magnesium, calcium and potassium salts dissolved in the pore fluids of Norwegian clays. Rather than chemically determining their concentrations, the specific electric conductivity of the pore fluid is measured, and a salt concentration quoted for the fluid an equivalent concentration of NaCl/l that possesses the same specific electric conductivity as the natural pore fluid.

and a detailed study of the profile has been made by Löken (1966).

The purpose of the tests on Sköyen clay was to begin with a natural salt clay, to leach it in the laboratory in the hope that a quick clay could be obtained, and during these processes to follow any changes of the radial stresses. It was expected that K_o for the salt clay would be about 0.5 but the question remained as to how much K_o would change due to leaching. Four tests were performed, the samples being leached at pressures lower than, equal to, and greater than P_o'. A typical result is given in Fig. 2-5. The sample was put into the testing apparatus and was subjected to a very small axial stress until it was certain that the K_o-membrane and the other electrical equipment were functioning properly. Axial loads were then applied at intervals and the axial compression and membrane strain were measured. At an axial stress of 0.42 kg/cm^2 ($\sigma_a'/P_o' \approx 0.4$) a seepage flow of salt water was begun through all the samples. The axial loads were then increased at time intervals of about 1 week until the axial stress was reached at which the sample was to be leached. Before leaching began each sample was allowed to stand for 2 weeks in order that the secondary-compression characteristics could be determined. The seepage water was then changed from 27 gm/l to 0.5 gm/l and seepage at constant axial stress was continued until the volume of leachate was at least 5 times greater than the pore volume of the sample and until the measured salt content of the leachate was less than 1 gm/l. Axial loading was then resumed and finally the samples were unloaded, removed from their membranes and routine tests performed.

An important detail concerning leaching was found in earlier tests, and that was that seepage water will always attempt to follow the more open zones in a sample and that these open spaces will be rapidly leached. The leachate will therefore often indicate small salt concentrations even though the concentration in the soil is considerably larger. Therefore it is necessary to have large seepage volumes, or more correctly to use much time, when leaching clays in order that salt located away from the more open seepage zones has the opportunity to diffuse towards the water of lowest salt concentration.

The results of the tests are given in Table 2-2 and Fig. 2-6, plotted in the form of a principal-stress diagram and a compression diagram. In the stress diagram it will be noticed that, for conditions of leaching at constant stress over time periods approaching 2 months, all tests indicated small increases of σ_r' and K_o but in no test did K_o exceed 0.6. From the result of test No. 222, given in Fig. 2-5, it can be concluded that a major part of the increase of K_o is directly due to the reduction of salt concentration in the pore fluid. The effect of leaching on the plasticity and undrained shear strengths of the samples is given in Table 2-2. The sensitivities of samples 222 and 224 were appreciably increased, being greater than 20 after the tests as compared to 5 before the tests. Although leaching produced very sensitive clays, it did not produce anything comparable to a typical quick clay. Thus, the conclusion that can be drawn is that for very sensitive clays formed by laboratory leaching, $K_o \leq 0.6$, with little change of K_o occurring during leaching.

Results of tests on Drammen clay.

These test results are presented in Fig. 2-6 to indicate the manner in which K_o changes with time for conditions of constant axial stress for a normally consolidated clay that exhibits pronounced secondary compression. The clay is an insensitive salt clay from Drammen and some of its classification properties are given in Table 2-2. The test results indicate that for the salt clay $K \approx 0.5$ and that during rest periods of constant axial stress when large secondary compression is occurring, there is only a very slight tendency for K_o to increase. This information is given as evidence (but not as proof) against the possibility that slow secondary settlements will lead to, or are manifestations of, important increases of radial stress and K_o with time.

4. K_o Measurement in the Field.

The picture that developed from the laboratory measurements of K_o was that $K_o \approx 0.5$ to 0.6 represented the range of values that could be obtained in the laboratory for reconsolidated quick clays, insensitive salt clays, and very sensitive leached clays. Although the results were very

consistent and reproduceable, there were shortcomings in the test procedures and apparatus and also there was the important question of the effect of geologic time, a factor that could not be studied in a laboratory. The alternative and quite different method was to attempt to measure in the field, with the use of some measuring device, the in situ horizontal stresses in clay deposits. Because the main interest was in quick clays, the first 3 sites that were investigated were in quick clays.

Apparatus.

The main requirement of the measuring device was to measure total horizontal earth pressures and water pressures at various depths in a clay deposit by installing the apparatus in the ground without disturbing the clay or altering the in situ stress condition. As it was impossible in practice not to disturb the clay, an apparatus was sought that would cause as little disturbance as possible.

Many ideas were considered, among which was the use of a single plate instrumented with pressure gauges that would be thrust vertically into the ground; this would have been a displacement device. Another idea was to use two instrumented plates forming the sides of a narrow box in which would be installed cutting blades and a flushing system; this would have been a non-displacement device. The idea that was finally adopted was the result of field experience with the installation of large open pipes in quick clays for the purpose of casting in place large-diameter concrete piles. On one such job these open pipes were to be installed close to existing buildings founded on footings and there was some concern that the pipes would displace the clay outwards, causing disturbance to the quick clay and settlement of the buildings. To study the problem a metal plate in the form of a ring was placed on the ground surface, and a small plate was placed inside the ring. The pipe was then allowed to sink into the soil between the plate and the ring. Level measurements on the two plates indicated that there was no measurable heave outside the pipe, whereas a large heave occurred inside the pipe, as a result of the entry of a volume of clay equal to the volume of the pipe. This experience indicated that an instrumented open pipe would not act as a displacement device and would in all probability cause as little disturbance to

the clay as more complicated and costly instruments.

The finished instrument is shown in Fig. 2-7. It consisted of steel-pile sections, Krupp KP 24, 47 cm wide, 1.5 to 3 metres in length and bolted together to give as even an outside surface as possible. The bottom edge of the pipe was bevelled so that soil would be cut and pressed into the pipe rather than to the outside. On the flat surfaces of the pipe were installed earth-pressure and water-pressure gauges of the type shown in detail in Fig. 2-7 (Øien, 1962; Bjerrum, Kenney and Kjærnsli, 1965). The gauges contained vibrating-wire strain gauges, and measurements were taken electronically from the ground surface by means of a frequency counter. The earth and water pressures were determined from the frequency measurements by using the results of pressure calibration tests on the gauges.

Similar to all mechanical gauges, the accuracy of the vibrating-wire gauge depends on the stability of the zero-pressure reading (zero reading) and on the stability of the slope of the calibration curve, in this case a curve of frequency against applied pressure. Experience has indicated that the slopes of the calibration curves for these gauges are very stable and significant variations have only been obtained when the membrane of the gauge has been damaged (Øien, 1962). However, with the gauges used for the K_o-instrument, and more specifically, with the gauges that were in contact with quick clay (earth-pressure gauges), there occurred important changes of the zero reading. From the gauge readings taken when the pipe was in the ground (these readings will be dealt with in detail in a later section), it was found that the measurements varied smoothly with time with no sudden changes, and therefore it was possible to conclude that the zero-reading changes did not occur abruptly during the measurement period. There are three likely possibilities of when the changes did occur: (i) the zero readings could have changed suddenly when the pipe was installed in the ground, or (ii) suddenly when the pipe was withdrawn from the ground, or (iii) they could have changed gradually during the measurement period. Because the cause of the zero-reading changes is not known it is impossible to conclude when the changes occurred, but it is perhaps most probable that the changes occurred gradually rather than suddenly.

At all three measurement sites the water-pressure gauges did not perform entirely satisfactorily due to the fact that they were not successfully saturated before the apparatus was installed in the ground. As shown in Fig. 2-7 there exists an open space between the gauge membrane and the filter plates and any air trapped within this space and within the filter plates would act as a damper on the performance of the gauge, the air being compressed for increases of water pressure and expanding for decreases of water pressure. This would lead to underestimates of the correct water pressures during periods when the measurements indicated increasing water pressures, and to overestimates during periods of measured decreases of pressures. During periods when the water-pressure measurements are constant the measured pressures are equal to the actual water pressures in the soil.

Although the apparatus was built to measure in situ stresses for conditions of zero deformation, unavoidable disturbance of the soil along the surface of the pipe will always cause the effective stress transmitted against the measuring device to be smaller than the original in situ horizontal effective stress for the case of the measuring device not displacing the clay outwards. The effect of clay disturbance is indicated in Fig. 2-8. Element A is located at a sufficient distance from the pipe to be completely unaffected by it, and it follows that the horizontal effective stress acting on element A is the stress that we wish to measure. Element B is located at the surface of the pipe, and the total earth pressures transmitted by this element and the water pressures in this element are those measured by the gauges on the pipe. Two conditions will be considered, the first of which is that there is zero disturbance of the clay in contact with the pipe. In this case element B has not been affected by the pipe and the stresses acting in it will be exactly equal to those in element A. Therefore the pressure measurements made on the surface of the pipe at element B will be equal to the true in situ stresses. In the second condition a zone of completely remoulded quick clay (fluid) is formed at the surface of the pipe. At time zero, immediately after the pipe is placed in the ground, the effective stresses in this zone are equal to zero

and the total stress is taken by pore-fluid pressure. Thus, at time zero in the stress-time diagram, σ_B' is equal to zero and $u_B = \sigma_B > \sigma_A$ because:

$$u_B \geq \gamma_s h \quad \text{(from field measurements presented in the following sections)}$$

and
$$\sigma_A = \gamma_w h + K_o(\gamma_s - \gamma_w)h$$
$$= \gamma_s h - (1-K_o)(\gamma_s - \gamma_w)h$$

where γ_s and γ_w are the densities of soil and water, and h is the distance to the ground surface. With the passage of time the water pressure at element B decreases due to drainage until ultimately it becomes equal to the original water pressure before pipe installation. Also, as the result of drainage, the remoulded clay around the pipe consolidates and the undisturbed clay adjacent to the remoulded clay swells, and because these are radial strains the hoop stresses in the undisturbed clay that swells will increase and the radial stress transmitted to element B will decrease. This phenomenon is commonly called arching and it causes the ultimate values of σ_B and σ_B' to be smaller than σ_A and σ_A', and therefore the values of earth pressure measured on the surface of the pipe will be smaller than the original in situ values.

From these considerations the following conclusions can be drawn. Because it is inevitable that some disturbance of the clay in contact with the pipe will occur, any radial effective stress measured on the surface of the pipe will be smaller than the original in situ value. It can be anticipated that if the thickness of the remoulded zone is very small, the excess water pressures will dissipate rapidly, the radial effective stress will increase rapidly to its ultimate value, and the underestimation of the true σ_r' will be small. Thicker zones of disturbance will lead to larger underestimates of true σ_r'. Another conclusion is that soil disturbance leads to reconsolidation and to radial strains, violating the definition of K_o conditions. Because soil disturbance certainly occurred during the installation of the pipe at the 3 sites to be described in the following sections, the ratio of radial to vertical effective stresses will be referred to as K, rather than K_o.

Measurements at Tönsberg.

General. The test site at Tönsberg was a flat, low-lying area slightly above sea level at the base of a steep, open-jointed rock escarpment (see photo in Fig. 2-7). A typical soil profile is given in Fig. 2-9; the clay is quick, slightly plastic ($I_p \approx 15\%$) and has large water contents. The surface weathered zone is not much more than 2 metres thick and the underlaying clay has an undrained shear strength of less than 2 t/m^2. Two piezometers at depths of 4 and 12 metres indicated the existence of overpressures and these probably were the result of water pressures in the rock being transmitted to the clay. The existence of water overpressures is also an indication that the clay exists today at reduced vertical stresses compared to the stress conditions shortly before the area was isostatically raised above sea level.

The locations of the instruments on the pipe are shown in Fig. 2-9. Each gauge level was 25 cm from the end of the pipe section to which the gauges were fastened and gauge level A was 25 cm from the bottom end of the pipe. Four earth-pressure gauges were mounted at levels A and C to investigate if equal pressures acted against the four sides of the pipe. Two water-pressure gauges were mounted at levels B and D.

A pit about 1.5 metres deep was excavated into the surface weathered clay and water flowed freely into the excavation through cracks to a level 1.2 metres from the ground surface. A ring and a plate were placed in the excavation and level measurements were taken to check the surface movements of the clay. The pipe was brought into place by a crane and allowed to sink into the ground by its own weight, which it did except for the final 2 metres when the weight of 3 men was used to keep it in motion. It stopped abruptly when it contacted a silty or sandy layer. The whole descent was completed within one or two minutes. Elevation determinations showed that the ring outside the pipe had not moved and that the plate inside the pipe had risen, the volume of additional clay in the pipe being within 5% of the estimated volume of the pipe, gauges and cables.

Readings of the gauges were taken before the pipe was lifted into the air and also when it was hanging vertically in order to check that all the

gauges were operating and that there had been no change of the zero-pressure readings. After the pipe was in the ground, frequent measurements were taken during the first few days, after which readings were taken less frequently. At the conclusion of the measurement period the pipe was extracted and the gauges were recalibrated. The slopes of the calibration curves were found to be unchanged but zero-pressure frequencies had changed, and these frequency changes are listed in Table 2-3 together with their corresponding pressure changes.

Water-pressure measurements. As can be observed in Table 2-3 the water-pressure gauges remained stable and there was no problem in interpreting the readings. The pressures are plotted against time in Fig. 2-9. During the first few days the pressures increased and then they decreased, in marked contrast to the earth-pressure measurements that began as large values and decreased steadily with time. The water pressures in fact must also have begun as high values, due to the soil disturbance resulting from the insertion of the pipe, but the response lag of these gauges was no doubt caused by the gauges not being saturated before installation. Of interest is the observation that the water pressures had about reached equilibrium 10 days after the pipe was put into the ground.

The final water pressures measured at the surface of the pipe have been plotted together with the measurements made with piezometers U. 1 and U. 2 to give a water-pressure profile of the site. The distribution is called Case 1 and is denoted on the drawing by u 1. A second distribution, Case 2, has been drawn for water pressures increasing hydrostatically from the ground surface, this corresponding to the conditions at the site prior to emergence from below sea level and prior to the existence of post-consolidation overpressures as found today. This is denoted by u 2.

Earth-pressure measurements. Although the earth-pressure gauges were in the ground for a period of only 2 months, large zero-point changes occurred and large uncertainties concerning the measured earth pressures resulted (Table 2-3). At level A the zero-point changes corresponded to pressure changes of 0.8 t/m^2 (0.7 to 0.9) and at level C they corresponded to pressure changes of 1.3 t/m^2 (from 0.9 to 1.5 t/m^2, neglecting gauge

6.E that was faulty). Because it was not known at what time or at what time rate these changes occurred, the measurements were dealt with in the following manner.

1. Pressures were calculated using the zero readings obtained at the end of the job. These results are plotted in Fig. 2-9, and it is seen that three of the four gauges at each of levels A and C gave essentially the same results. One gauge at each level (gauges 4.E and 6.E) gave appreciably different results and these gauges will be neglected.

2. Because of changes of the zero points, the values of estimated earth pressure were overestimates by amounts of 0 to 0.8 t/m^2 at level A and 0 to 1.3 t/m^2 at level C, the largest errors probably applying to the earliest measurements. Account of these errors has not been taken in the plot of total earth pressure against time in Fig. 2-9, but account will be taken of them when radial effective stresses are estimated.

Radial effective stresses σ_r' were calculated by estimating the water pressures at the levels of the earth-pressure gauges and subtracting these pressures from the measured earth pressures. At each gauge level two earth-pressure estimates were made: the first estimate was based on the final calibrations and probably gives accurate values for the end of the job but probably seriously overestimates the earth pressures at the beginning of the job, and the second estimate was based on the assumption that the zero readings varied directly with time from the values measured before and after the job, and this estimate possibly gives more reasonable estimates of the earth pressures throughout the job.

The vertical effective stress acting at a gauge level was taken as being equal to the vertical total stress due to the weight of the soil minus the estimated water pressures. Values of K, the ratio of the measured radial to vertical effective stresses, are presented in Fig. 2-9. The uncertainties concerning the estimated earth pressures and water pressures make it unwise to conclude anything concerning the manner in which σ_r' and K varied during the job. However, the accuracy of the measurements near the end of the job should be good, and these results indicate that at level A (depth 7.4 metres) the stress ratio was $K \approx 0.65$, and at level C (depth 3.9 metres) the

K-value was 0.59. For the condition of zero overpressures (Case 2, normal consolidation) the K-values were 0.5 at level A and 0.6 at level C.

Measurements at Manglerud III.

General. The test site at Manglerud III was a flat, very wet and boggy area in the outskirts of Oslo. The ground-surface elevation is 110 metres, and this indicates that it rose above sea level about 9000 years before present (Kenney, 1964) and therefore the near-surface clays at this site are appreciably older than those at Tönsberg where they have only in recent geologic time risen above sea level. A typical soil profile is given in Fig. 2-10; the site is covered with a 2-metre layer of organics, below which is a strong weathered zone about 2 metres thick overlaying some 20 metres of sensitive silty clay. The silty clay has undrained shear strengths of about 1 t/m^2 and plasticity indices about I_p = 12%, and it is both weaker and less plastic than the Tönsberg clay. Prior to installing the pipe, this site had been thoroughly explored and it was known that the water surface was at the ground surface and that the water pressures increased nearly hydrostatically with depth, being only several decimeters greater than hydrostatic at 10 metres and 2 metres greater than hydrostatic at 20 metres.

Several changes were made of the locations of the gauges on the pipe in comparison with the arrangement used at Tönsberg. Rather than concentrating earth-pressure and water-pressure gauges separately at the various gauge levels, two earth-pressure and two water pressure gauges were put at levels A and C and two earth-pressure gauges each at levels B and D. Another change was to place a 1.5-metre length of pipe below the lowest level of gauges in order to remove these gauges from any stress complexities in the clay near the bottom end of the pipe.

The pipe was ready to be put into the ground in about mid-November but a very early winter set in and the temperature fell below $0^{\circ}C$. It was feared that by trying to put a very cold pipe into the ground, the soil coming into contact with the pipe would freeze, adhere to the pile and thus increase greatly the effective thickness of the pipe and cause considerable soil disturbance as the pipe sank into the clay. The job was therefore delayed until the first mild day, which did not arrive until early December. At this date

time was running short to get the pipe installed, and it was decided to put the pipe in the ground regardless of soil disturbance, simply because it probably was the last opportunity before real winter set in.

A shallow pit had been dug into the organics but it was not practical to extend it into the weathered clay below. When the pipe was lowered it sank under its own weight through the organics but would not penetrate through the weathered clay. It was raised and lowered four times and each time an additional 1-2 metres penetration was obtained, but each time the pipe was raised there was clay adhering to its surface, indicating that in all likeli - hood a layer of weathered clay was adhering to the pipe over its full length and that soil was being removed from around the pipe at the upper levels. After the pipe was raised for the fourth time it sank under its own weight the final 4-5 metres of its final position. Gauge measurements were made at regular intervals during the winter. They were discontinued during the spring and early summer, and were resumed again in late summer shortly before the pipe was extracted.

Except where noted otherwise in Fig. 2-10, the pressures measured during the first 90 days were determined by using the zero readings obtained before the pipe was installed, and the pressures measured during the month prior to extracting the pipe were determined by using the zero readings obtained after the pipe was extracted. It is of interest that three of the four water pressure gauges experienced no change of zero reading over the test period of 283 days whereas many of the earth-pressure gauges experienced large changes of zero reading and had rather severe surface corrosion. A summary of the changes of zero reading and the equivalent pressure changes is given in Table 2-3.

Pressure measurements. These results, in some respects, are similar to the Tönsberg measurements. The water-pressure gauges were not saturated and therefore exhibited time lags. Therefore, during the first 20 days when the water-pressure readings were changing rapidly it is reasonable that the water-pressure measurements, and therefore the effective-stress determinations, were slightly inaccurate.

The measurements taken of the earth-pressure gauges at level D require comment. As shown in Fig. 2-10, these pressures, one month after the pipe was installed, had decreased and had become equal to the hydrostatic pressure at this level. The measurements remained constant at this value until the end of the test and there seems to be little doubt that the measurements were of water pressure, this meaning that the undisturbed soil had fully arched over the gauges or perhaps had fully arched around the pipe, transmitting no measurable earth pressures to the gauges.

The radial effective stresses σ_r' were determined directly for levels A and C, and for level B they were determined from water pressures estimated by interpolation between the water-pressure measurements made at levels A and C. After the first 20 days when the water-pressure gauges became reliable, the σ_r' values for levels A and B remained rather constant throughout the test period. At level C, within the depth of clay where excess disturbance was probably caused during the installation of the pipe, σ_r' varied somewhat during the first 90 days and finally became constant.

The calculated K-values indicate that at levels A and B there was a slight tendency for K to decrease with time, perhaps due to reconsolidation of disturbed clay, but such small changes could also be due to instrument inaccuracies. At level C, K apparently increased after Day No. 70, and this could be taken as an indication that the clay yielded towards the pipe. Although this is certainly possible, it should also be noted that the earth-pressure values for the first 90 days are based on measurements of only one gauge (3.E), and although the gauge appears to have remained stable according to the zero readings (Table 2-3), it is possible that these measurements are not representative of average conditions around the pipe. The final K value, however, is a dependable result, and the low value of $K = 0.28$ is strong evidence that arching of the clay has occurred around the pipe.

Measurements at Kjelsås.

General. Kjelsås is located in the outskirts of Oslo at elevation 170 metres. This area emerged from below sea level about 9500 years before present and is therefore the oldest clay of the three test sites. The clay is

layered, has low plasticity ($I_p \approx 7\%$), very small undrained shear strengths (from 0.4 to 0.8 t/m^2), and is quick. This clay is one of the weakest quick clays in Norway, having $s_u/p \approx 0.1$, and it provided the opportunity to measure K_o in a clay being at the lower limit of the undrained-strength scale. A boring profile is given in Fig. 2-11 together with the pressure measurements.

The locations of the gauges on the pipe were the same as for the Manglerud III site. The only changes that were made were that the junctions of the pipe sections were made watertight by the use of a lead-filler compound and that attempts were made to saturate the water-pressure gauges.

A pit was excavated to a depth of 1.5 metres and the ring and plate were placed on the bottom of the pit. The pipe was installed on 27 October, 1966. It sank very easily into the clay under its own weight and stopped abruptly at a resistant layer at a depth of about 10 metres. Elevation measurements on the ring outside the pipe and on the plate inside the pipe showed that there was no heave of the bottom of the pit outside the pipe but that there was a clay heave within the pipe that corresponded to the total displacement volume of the pipe.

Pressure measurements. The results are similar to those for the Tönsberg and Manglerud III sites. The main difference is that the earth and water pressures took much less time to reach equilibrium, becoming practically constant after only 5 days. This is believed to be primarily due to the silty and layered nature of the clay providing good drainage. It is not believed to be because less soil disturbance was experienced at this site than the other sites; there must have been some disturbance at level A because shortly after the pipe was put in the ground the water pressures rose at least 50% above their ultimate values. The water-pressure gauges were not fully saturated, as indicated by their response-lag at the beginning of the measurement period.

The pressures have been calculated from calibration tests and zero readings taken before the pipe was put in the ground; the pipe is not scheduled to be removed from the ground until the spring of 1967. The results for the various pairs of gauges at different levels indicate small

differences, with the exception of gauges 5.E and 6.E at level B. In comparison with the measurements taken at level A (see σ_r' values plotted in the soil-profile diagram), gauge 5.E appears to be measuring too small values.

The difference between water pressures at level C and level A equals the hydrostatic pressure corresponding to the vertical distance between the two levels. Hence, the water pressures vary approximately hydrostatically with depth along the pipe from an implied ground-water surface 1 metre below the ground surface.

Radial effective stresses were calculated for levels A and C from the direct measurements of total earth pressures and water pressures. At levels B and D the water pressures were estimated from those measured at levels A and C. It was found that σ_r' at the 4 levels became essentially constant after only 5 days.

The calculated values of K are constant after the initial 5 to 10 days. At level B, where it is believed that gauge 5.E is measuring too low, K-values are given based on both the average pressure for gauges 5.E and 6.E and on the pressure from only gauge 6.E; this latter value is believed to be more correct. K at level A is 0.66, and for the upper levels K decreases with decreasing depth. The minimum value of K at this site is 0.35 at level D.

Remarks Concerning K_o Measurement in the Field.

The results obtained from the 3 sites are summarized in Fig.2-12, in which values of K are plotted against depth below ground surface. From this figure two things are evident; first, in all cases K increases with depth, and, second, at the Manglerud III site where great difficulties were experienced in getting the pipe into the ground and where greatest disturbance of the clay along the pipe resulted, the lowest values of K were measured.

It is known that at the Manglerud III site the upper clay was disturbed to a greater extent than the lowest clay by raising and lowering the pipe, and therefore for this site it can be concluded that the low K values

measured at small depths are probably the result of disturbance of the sensitive clay along the pipe, reconsolidation of this disturbed clay and, as a result, decrease of normal stress against the pipe due to arching. With increasing depth K increased, probably because the clay around the pipe was less disturbed and consequently there was less reconsolidation and less arching of the clay around the pipe. At the Tönsberg and Kjelsås sites the pipe sank easily into the clay under its own weight and at these sites, considering the very high sensitivities of the clays, the thickness of the disturbed layer of clay must have been small in comparison to that at Manglerud III. It can also be stated with reasonable confidence that at all sites, due to irregularities of the pipe surface, the upper clays that were subjected to the greatest drag movements of the pipe were disturbed to a greater extent than the clays near the bottom of the pipe. The thickness of the disturbed zone should be largest at shallow depths and smallest at greater depths. It is reasonable, therefore, that the increase of K with depth at all sites was due to a decrease of the thickness of the disturbed zone of clay with depth, and the reason why the K values measured at Tönsberg and Kjelsås are larger than those at Manglerud is that the degrees of soil disturbance at these sites were less than those at Manglerud. Another contributing factor to the decrease of K with depth at Manglerud III and Kjelsås could have been that the upper levels of gauges were in slightly weathered clay that could have arched to a greater extent than the lower unweathered clays.

There is at least one method of checking whether or not the measured increases of K with increasing depth are due to clay disturbance, and that is to concentrate the instrumentation near the bottom of the pipe and to lower the pipe into the ground in stages of about 3 metres, waiting at each new level until equilibrium is obtained. In this way pressure measurements are always made at the bottom of the pipe where least clay disturbance has occurred.

5. Conclusions.

1. K_o-consolidation tests performed in triaxial cells on samples of insensitive salt clays and natural quick clays gave approximately the same results; that is, $K_o \approx 0.5$.

2. Insensitive salt clay, K_o-tested in reinforced rubber membranes, gave the result $K_o \approx 0.5$. Similar clays that were leached and that became very sensitive (but not quick) as the result of leaching, gave slight increases of K_o, the values obtained being $K \leq 0.6$.

3. Measurements of in situ stresses at Tönsberg and Kjelsås in weak quick clays gave maximum values of $K = 0.65$. The maximum values of K were measured near the bottom of the instrumented pipe. Smaller values of K were measured at shallower depths at both sites, and it is probable that these lower values are the result of greater degrees of clay disturbance at the shallow depths. There is little doubt that the K-values measured near the bottom of the pipe have also been influenced by clay disturbance, but the magnitude of this effect is not known at this time. It is possible that by using concentrated instrumentation near the bottom of the pile, the variation of K along the pipe from the bottom edge (where disturbance is least) can be determined, and thereby a better estimate of K_o can be obtained.

TABLE 2-1. Properties of soils used in triaxial-cell K_o tests.

Sample No..	Field Data			Strength		Water Contents Before Test					Salt Content gm/l	After test		
	Hole No.	Depth m	P_o' t/m^2	s_u t/m^2	St	w %	w_L %	w_P %	I_P %	I_L		w %	s_u t/m^2	St
Manglerud III (F.264)														
37-2	1	7.5	4.5	1.2	60	50..6	32	22	10	2.9	1-2	42.7	0.9	-
37-3	1	7.3	4.3	1.2	60	49.7	32	22	10	2.8	-''-	35.3	3.4	7
45-8	2	7.6	4.6	1.2	60	47.2	33	22	11	2.3	-''-	42.7	0.6	32
45-9	2	7.5	4.5	1.2	60	47.1	33	22	11	2.3	-''-		0.6	21
43-10	2	5.7	3.2	1.4	35	46.2	32	22	10	2.4	-''-		1.3	6
Drammen (F.82)														
TX-2	I	5.9	5.6	2.2	10	53.3	52	33	19	1.1	22			
TX-6	I	7.4	6.5	2.3	10	51.9	65	29	36	0.6	26			
TX-1	I	8.7	7.4	2.5	9	51.8	54	27	27	0.9	27			
TX-5	I	11.9	9.6	2.0	3	34.1	39	23	16	0.7	27			

TABLE 2-2 Properties of soils used in K_o-tests with reinforced rubber membranes.

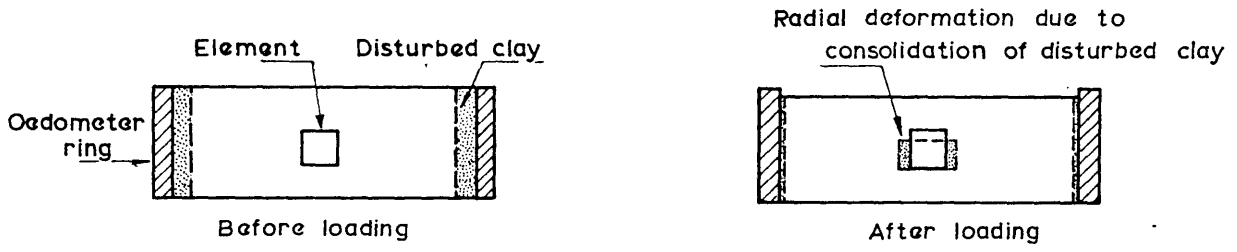
Sample No.	Field Data			Strength			Water Contents					Salt Content gm/l
	Hole No.	Depth m	P_o' t/m^2	s_u t/m^2	s_u^- t/m^2	St	w	w_L	w_p	I_p	I_L	
Sköyen (F.266)												
221	I	20.9	17.2	3.95	0.92	4.3	40.0	53.5	24.2	29.3	0.54	27.6
				8.10	1.55	5.2	39.5	46.0	25.9	20.1	0.68	1.0
222	I	21.0	17.3	4.50	0.91	4.9	40.5	53.1	25.8	27.3	0.54	
				3.35	0.25	13.4	41.6	-	-	-	-	1.1
223	I	21.1	17.3	4.09	0.92	4.4	41.4	50.9	22.8	28.1	0.66	27.6
				6.35	0.28	22.7	41.9	36.3	26.2	10.1	1.55	1.4
224	I	21.2	17.4	3.95	1.02	3.9	40.3	55.9	23.3	32.6	0.52	32.7
				6.01	0.68	8.8	39.3	43.0	28.0	15.0	0.76	1.1
Drammen (F.82)												
11	I	5.1	5.0	2.27	0.27	8.4	49.8	55.0	25.1	29.9	0.83	20
				4.0	0.80	5.0	44.0	53.8	26.8	27.0	0.64	
16	I	5.3	5.1	2.46	0.35	7.0	50.8	56.5	26.2	30.3	0.81	20
				5.87	0.85	6.9	43.5	54.4	29.1	25.3	0.57	

Note. For all tests, the first line is before-test results, the second line is after-test results.

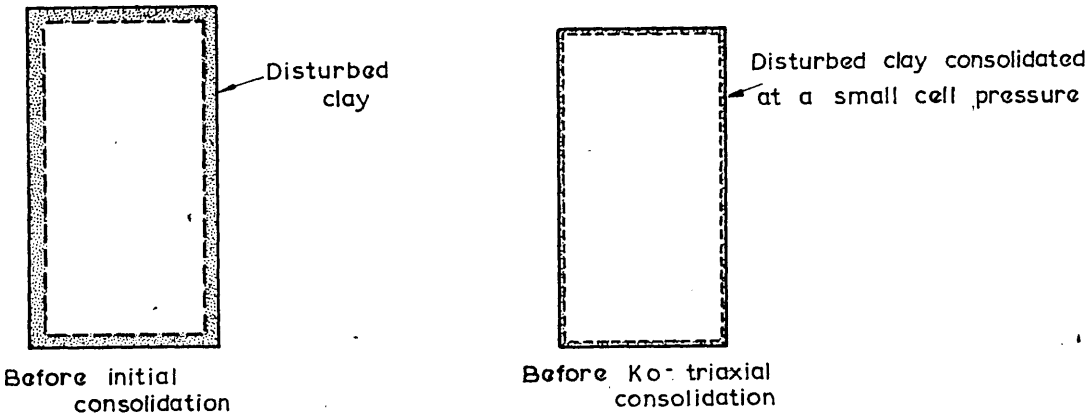
Table 2-2.

TABLE 2-3. Zero-reading changes and equivalent pressure changes of the gauges used at Tönsberg and Manglerud III.

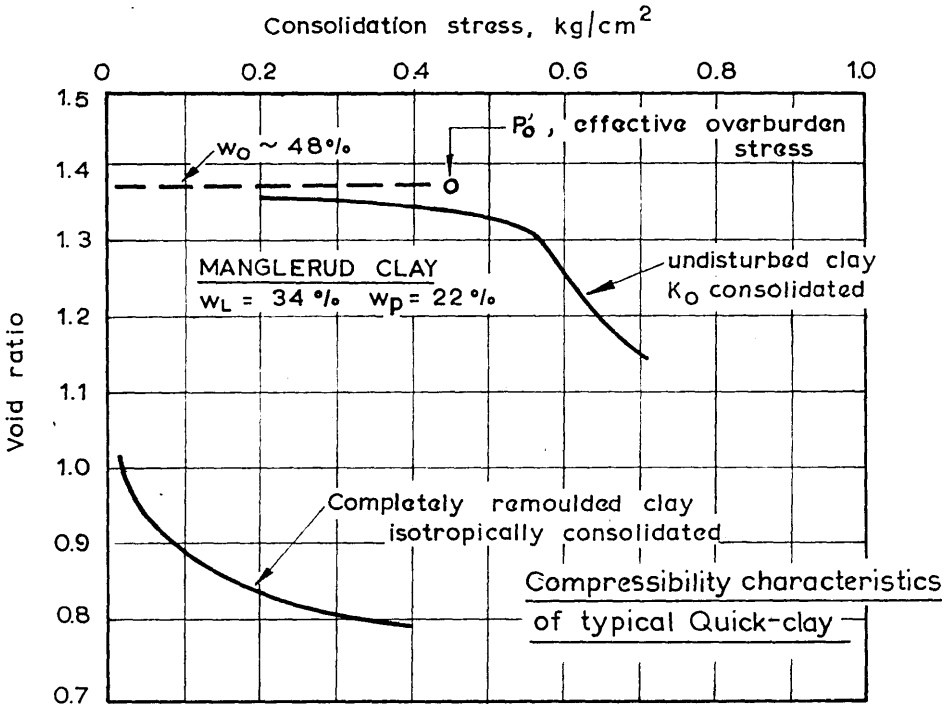
Gauge No.	Tönsberg			Manglerud III		
	Before 3/9-65	After 16/11-65	ΔP t/m^2	Before 10/12-65	After 20/9-66	ΔP t/m^2
1.E	1137	1089	0.9	1074	1004	1.1
2.E	1037	1002	0.7	987	979	0.2
3.E	1143	1108	0.7	1092	1101	0.2
4.E	976	928	0.8	911	794	1.9
5.E	928	818	1.5	806	823	0.3
6.E	1004	851	2.6	947	-	-
7.E	976	885	1.4	872	808	1.0
8.E	958	897	0.9	879	873	0.1
9.W	1013	1008	0	998	981	0.3
10.W	978	959	0.3	950	1009	1.1
11.W	981	980	0	970	975	0.1
12.W	966	969	0	952	971	0.3



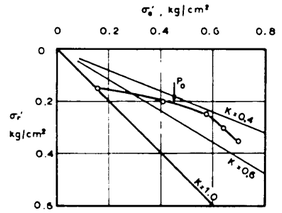
Oedometer test



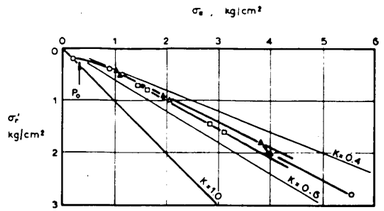
Ko-triaxial test

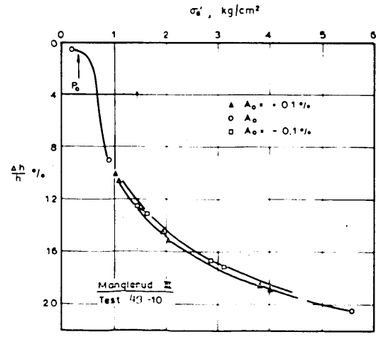


EFFECT OF SURFACE DISTURBANCE ON THE BEHAVIOUR OF QUICK-CLAY SAMPLES

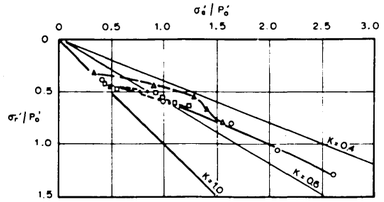


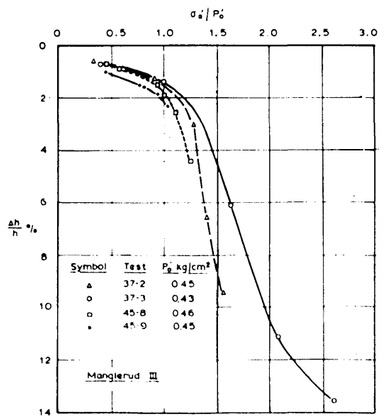
Typical Ko-triaxial test on quick clay



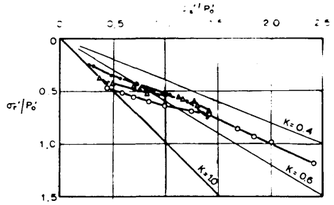


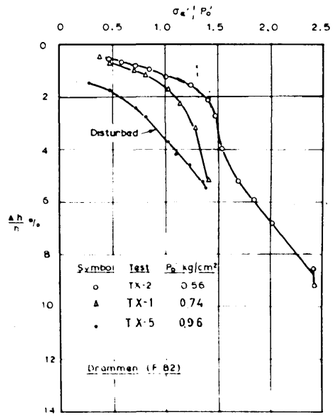
Ko-triaxial test with varying sample area





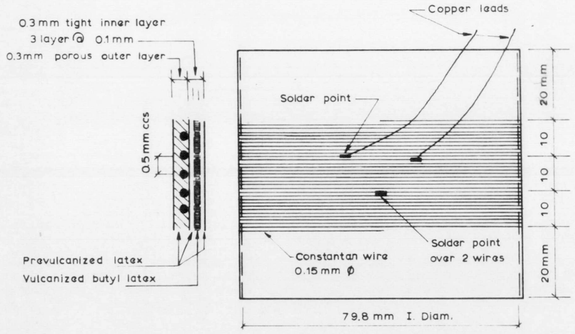
Results of Ko-triaxial tests on quick clay



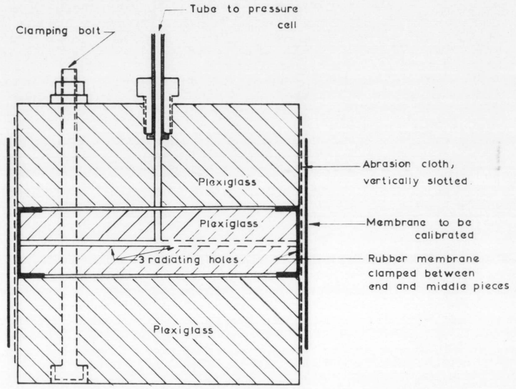


Results of Ko triaxial tests on insentive clay

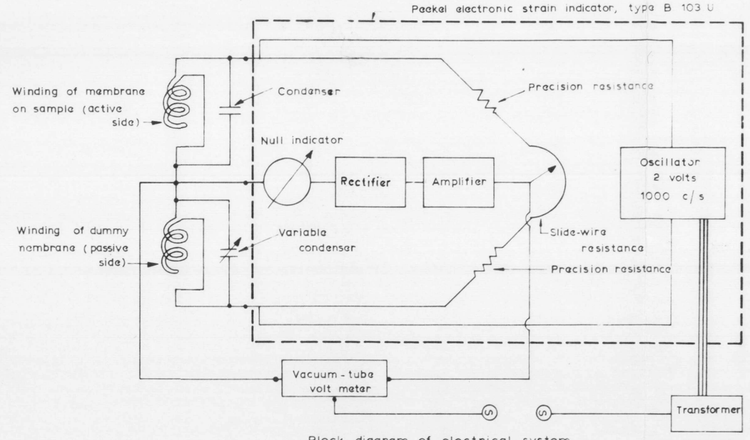
RESULTS OF TRIAXIAL-CELL K_o TESTS ON TWO CLAYS



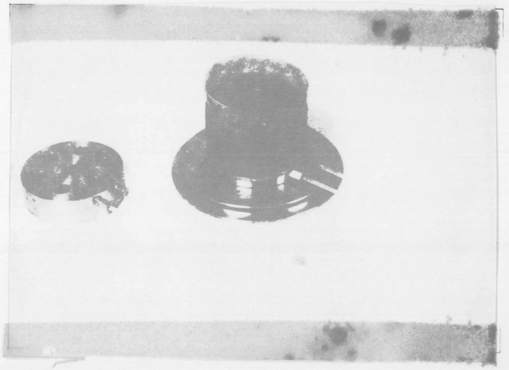
Details of membrane

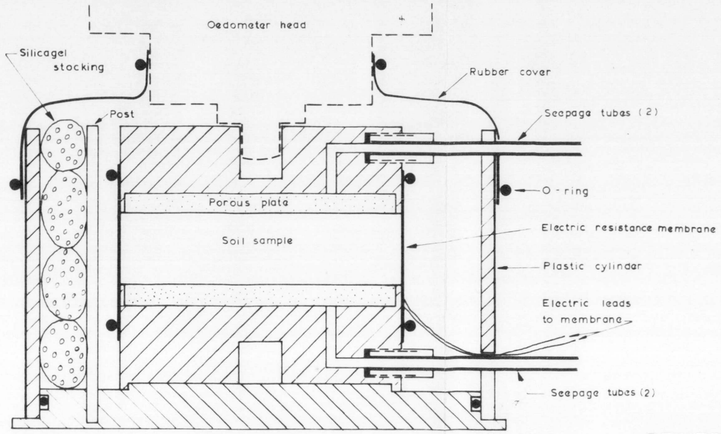


Membrane calibration cylinder



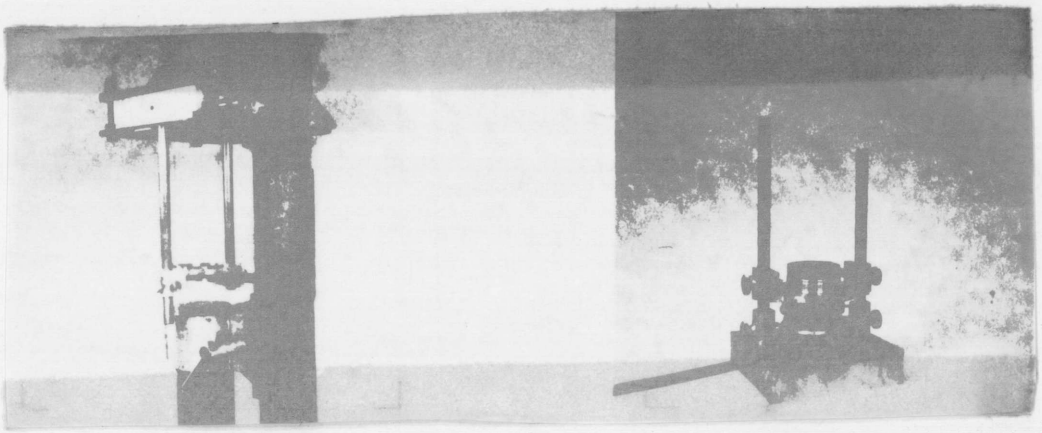
Block diagram of electrical system





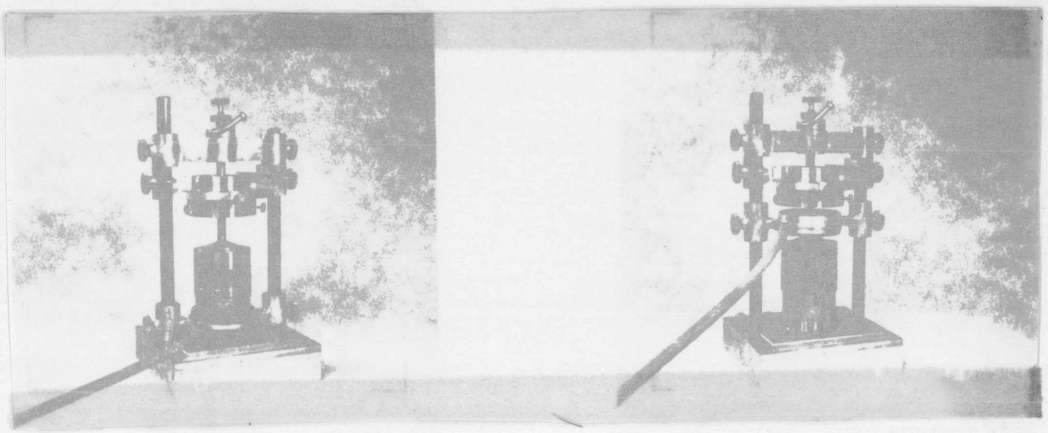
Test arrangement

REINFORCED RUBBER MEMBRANE FOR MEASURING RADIAL STRESS



1. Sample tube and cutting ring are fastened to extruder. Clay will be pressed into the ring in stages.

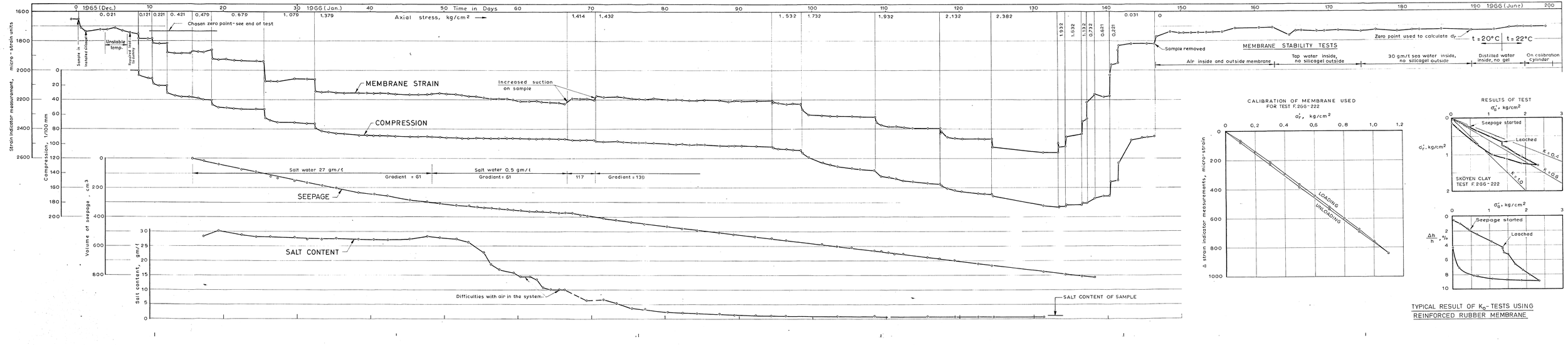
2. Cutting ring with sample, pedestal and vacuum cylinder with membrane are assembled together. Cutting ring has been lowered and sample will be cut to the required height.



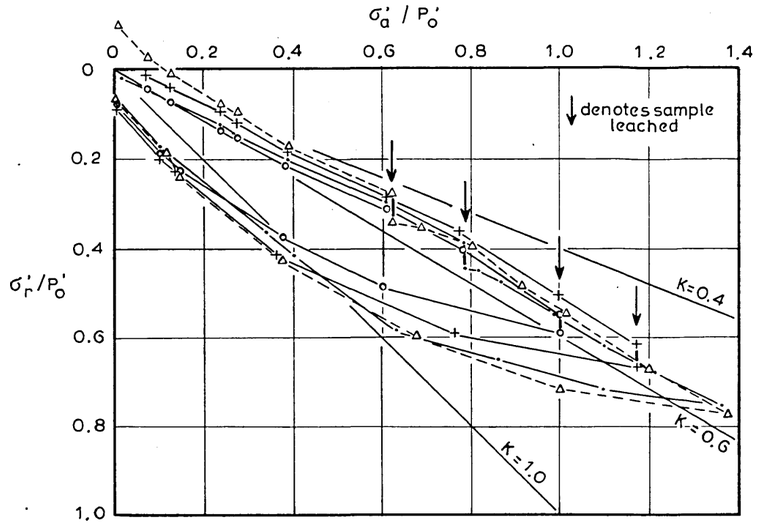
3. Sample is contained between top and bottom filter pieces. Vacuum cylinder will be raised and the membrane will be transferred to the sample.

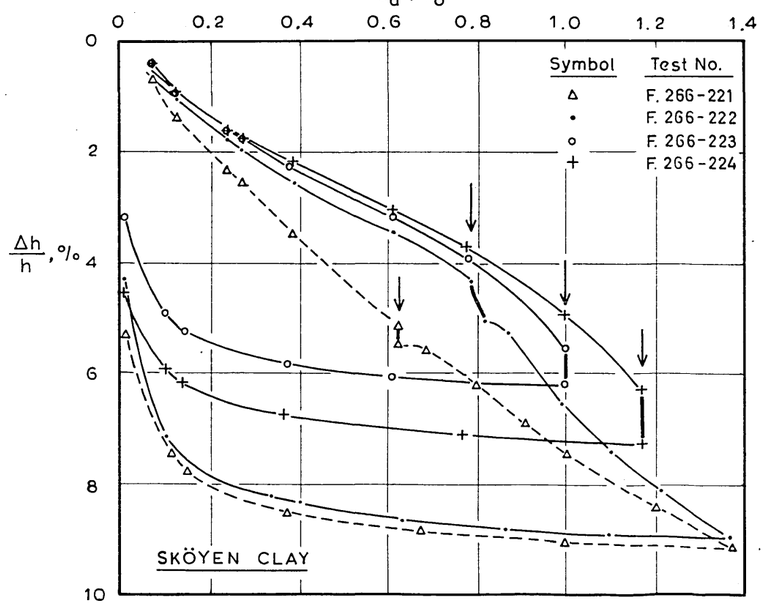
4. Membrane is in place and the vacuum cylinder raised. The sample is ready to be transferred to the testing device.

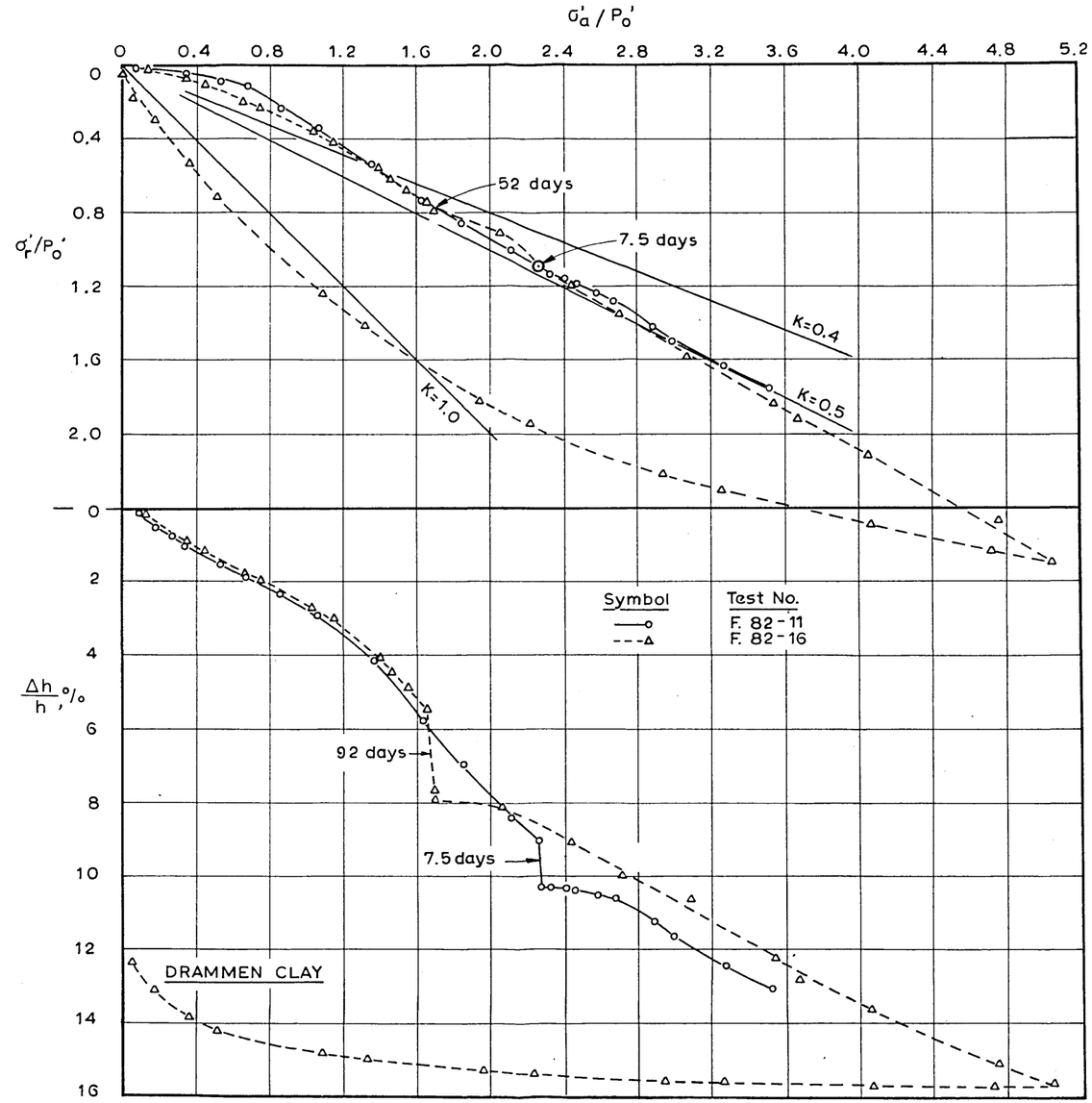
APPARATUS FOR CUTTING THE TEST SPECIMEN AND FOR MOUNTING THE REINFORCED RUBBER MEMBRANE



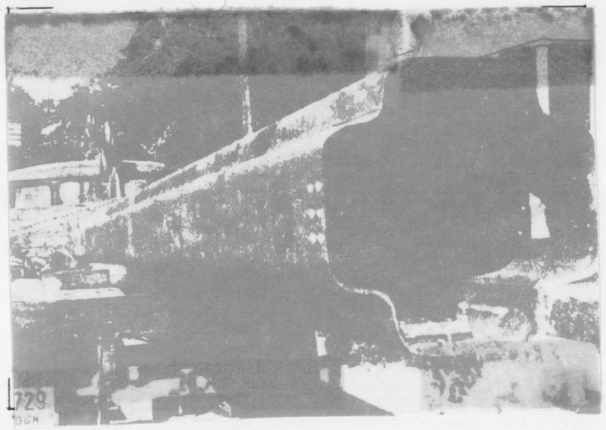
TYPICAL RESULT OF K_o-TESTS USING REINFORCED RUBBER MEMBRANE







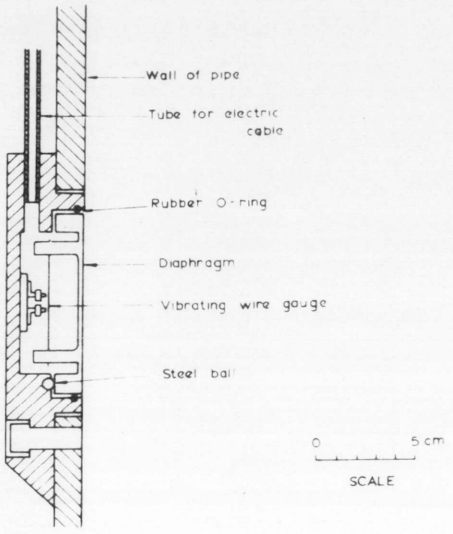
RESULTS OF K_o TESTS ON TWO CLAYS
USING REINFORCED RUBBER MEMBRANES



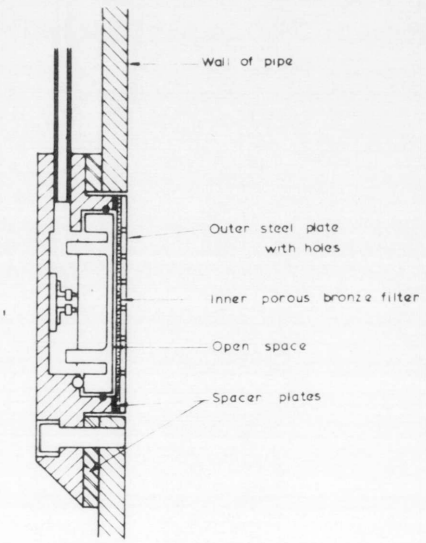
BOTTOM VIEW



APPARATUS AT TÖNSBERG

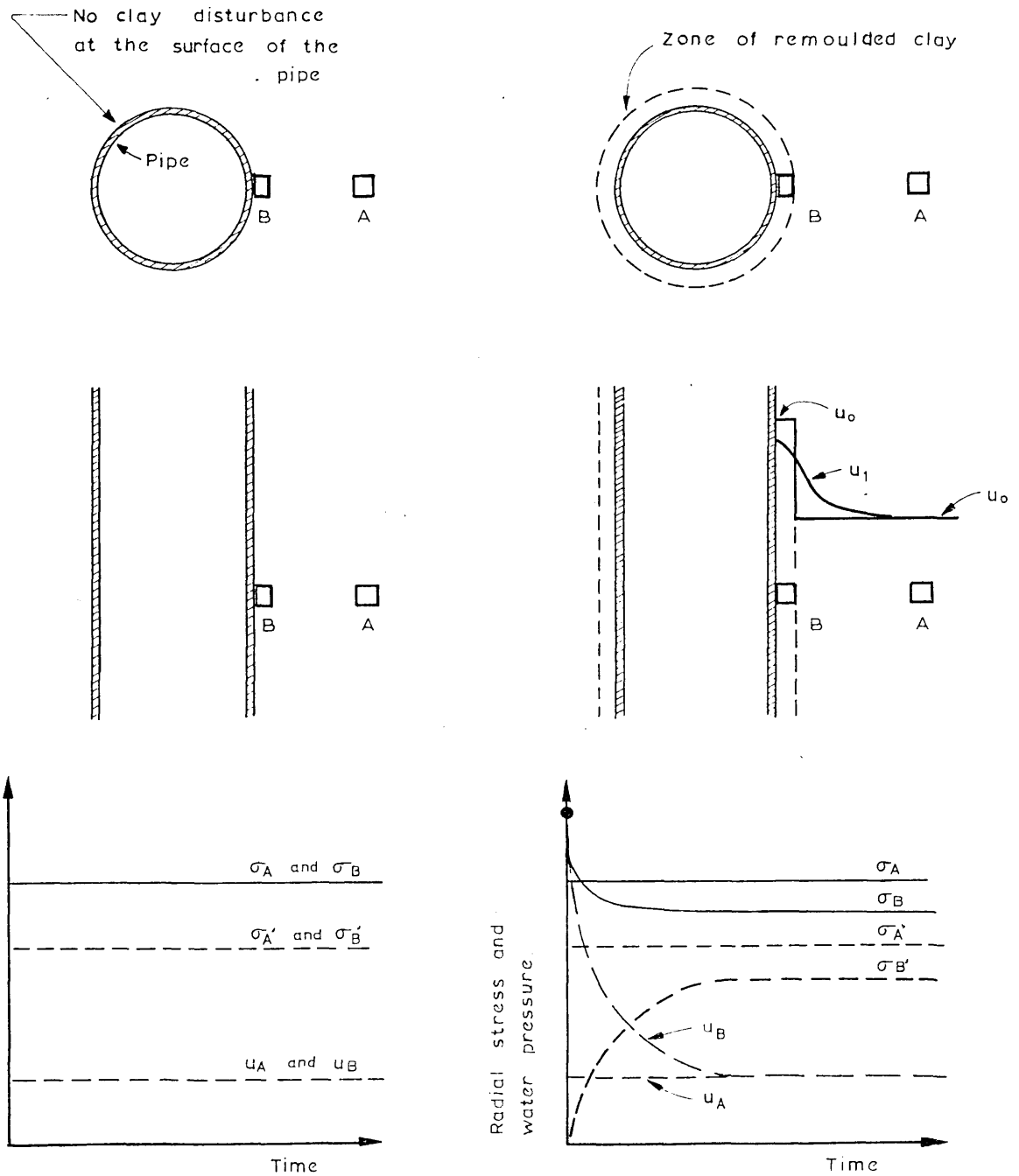


EARTH-PRESSURE GAUGE



WATER-PRESSURE GAUGE

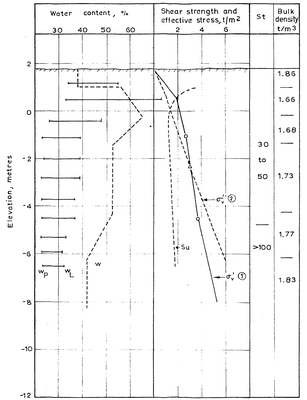
APPARATUS FOR MEASURING IN SITU HORIZONTAL STRESSES IN QUICK CLAY



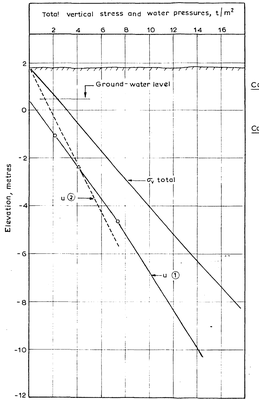
CONDITION 1. ZERO DISTURBANCE

CONDITION 2. DISTURBANCE

EFFECT OF CLAY DISTURBANCE ON IN SITU STRESS MEASUREMENT



A SOIL PROPERTIES

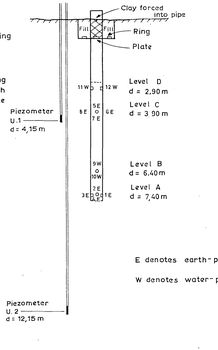


B. PROFILES OF STRESS

Notes

Case ① Water pressures corresponding to ground-water level and water-pressure measurements.

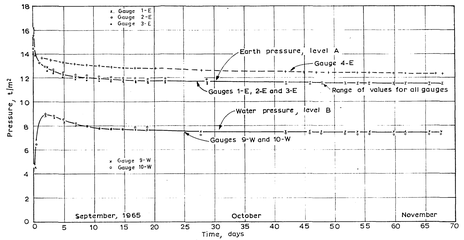
Case ② Water pressures increasing hydrostatically with depth from the ground surface

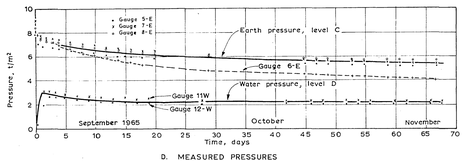


E denotes earth-pressure gauge

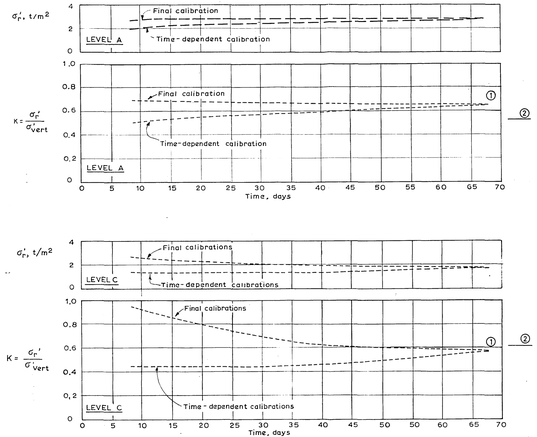
W denotes water-pressure gauge

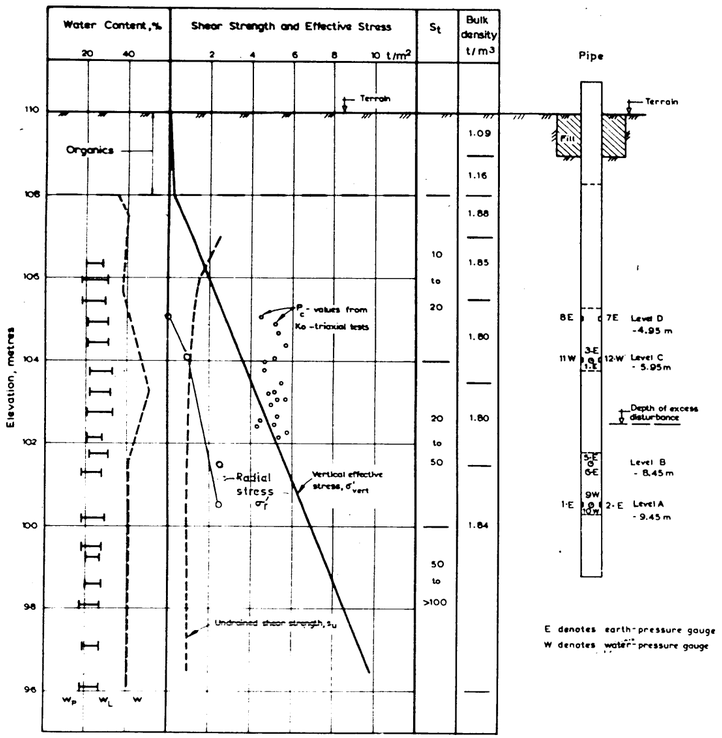
C. LOCATION OF GAUGES

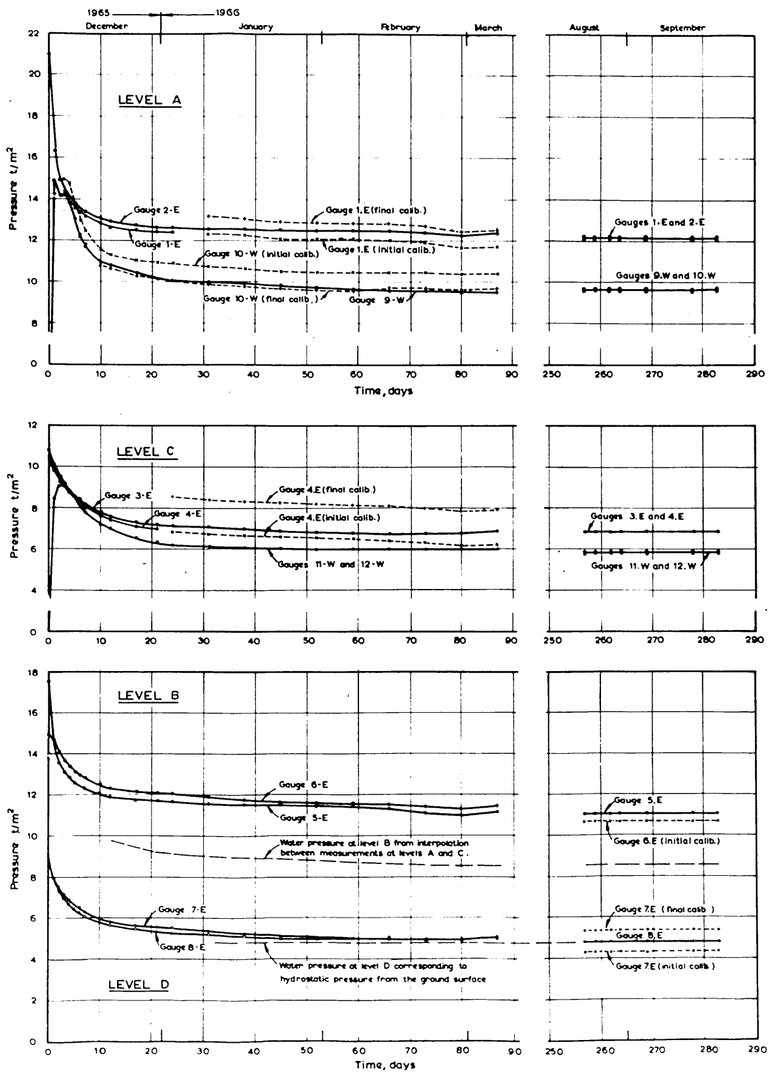


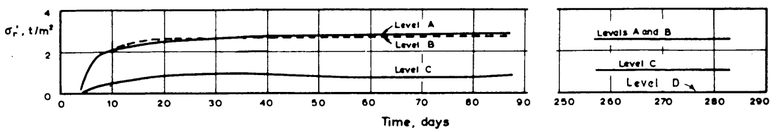


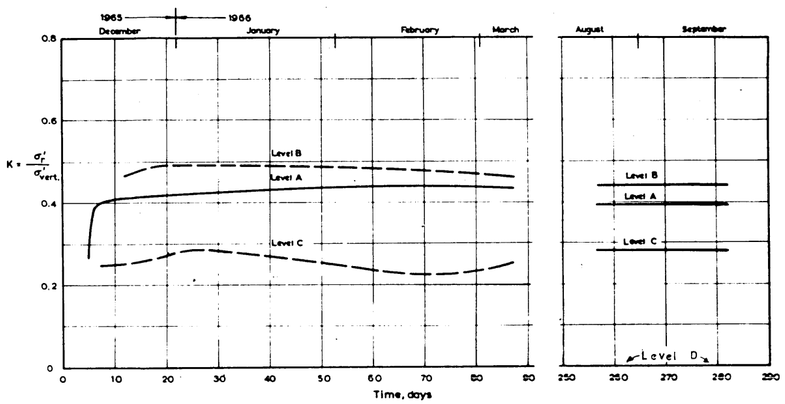
D. MEASURED PRESSURES











Legend

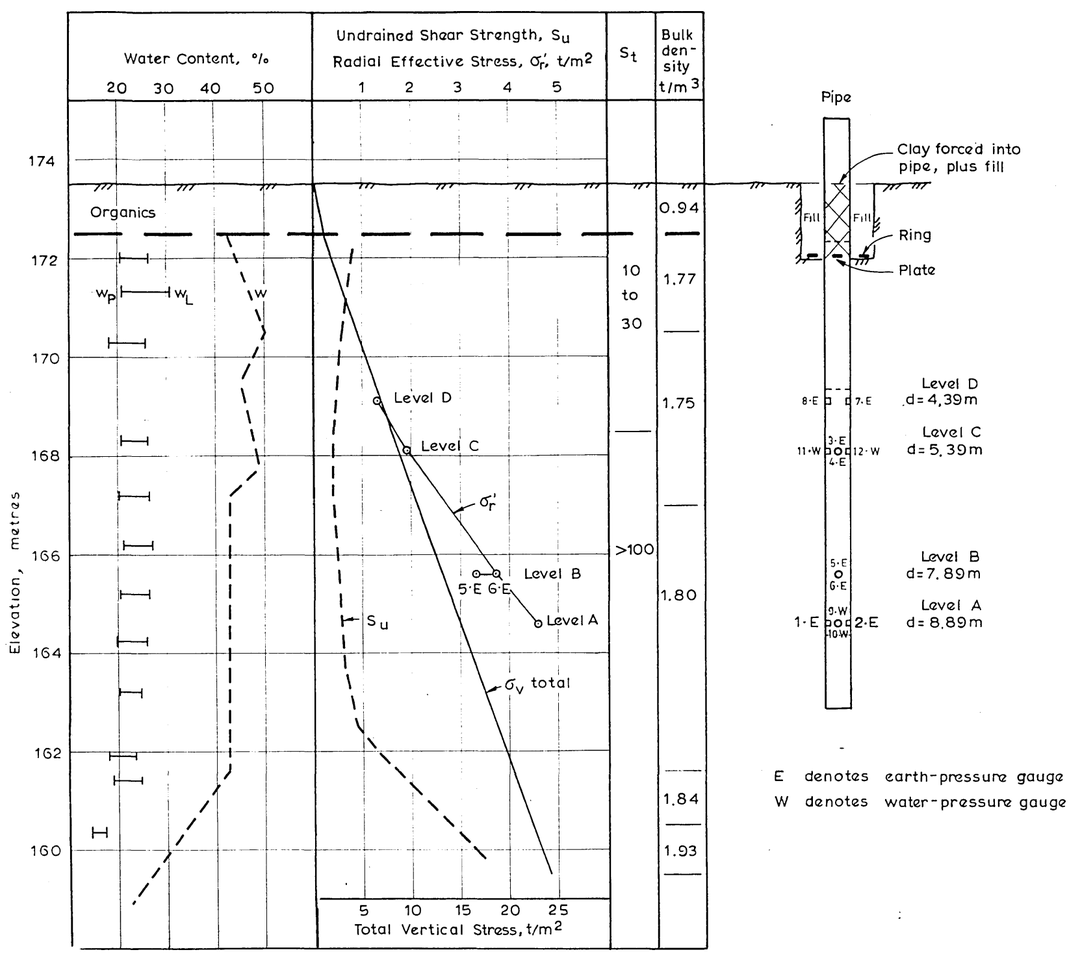
- Dependable results
- Undependable results

Notes

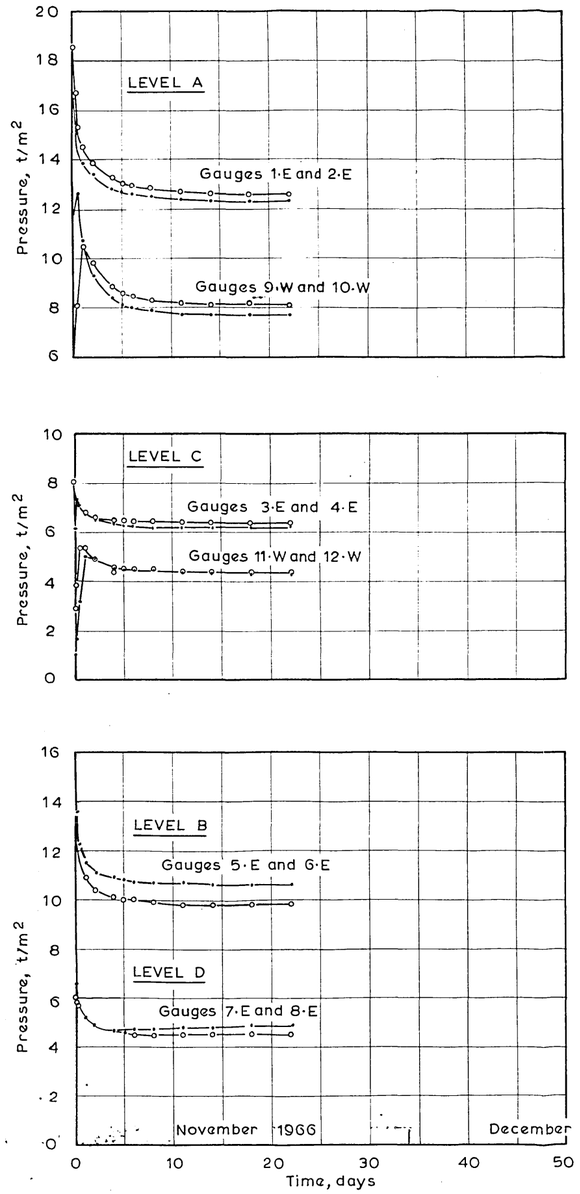
Measurement taken after Day 250 have been interpre ted on the basis of the final gauge calibrations, unless otherwise noted.

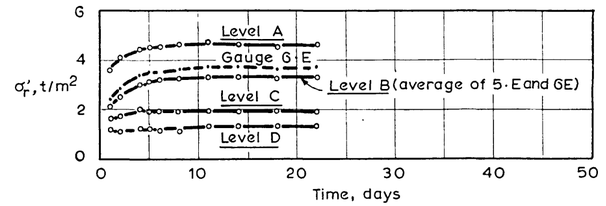
Gauge 6-E became inoperative after the pipe was removed from the ground and before it was calibrated

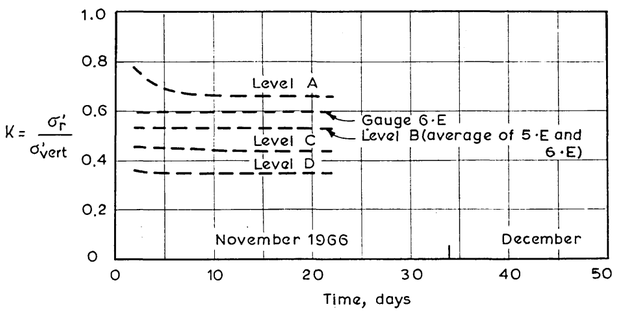
IN SITU STRESS MEASUREMENTS AT MANGLERUD III



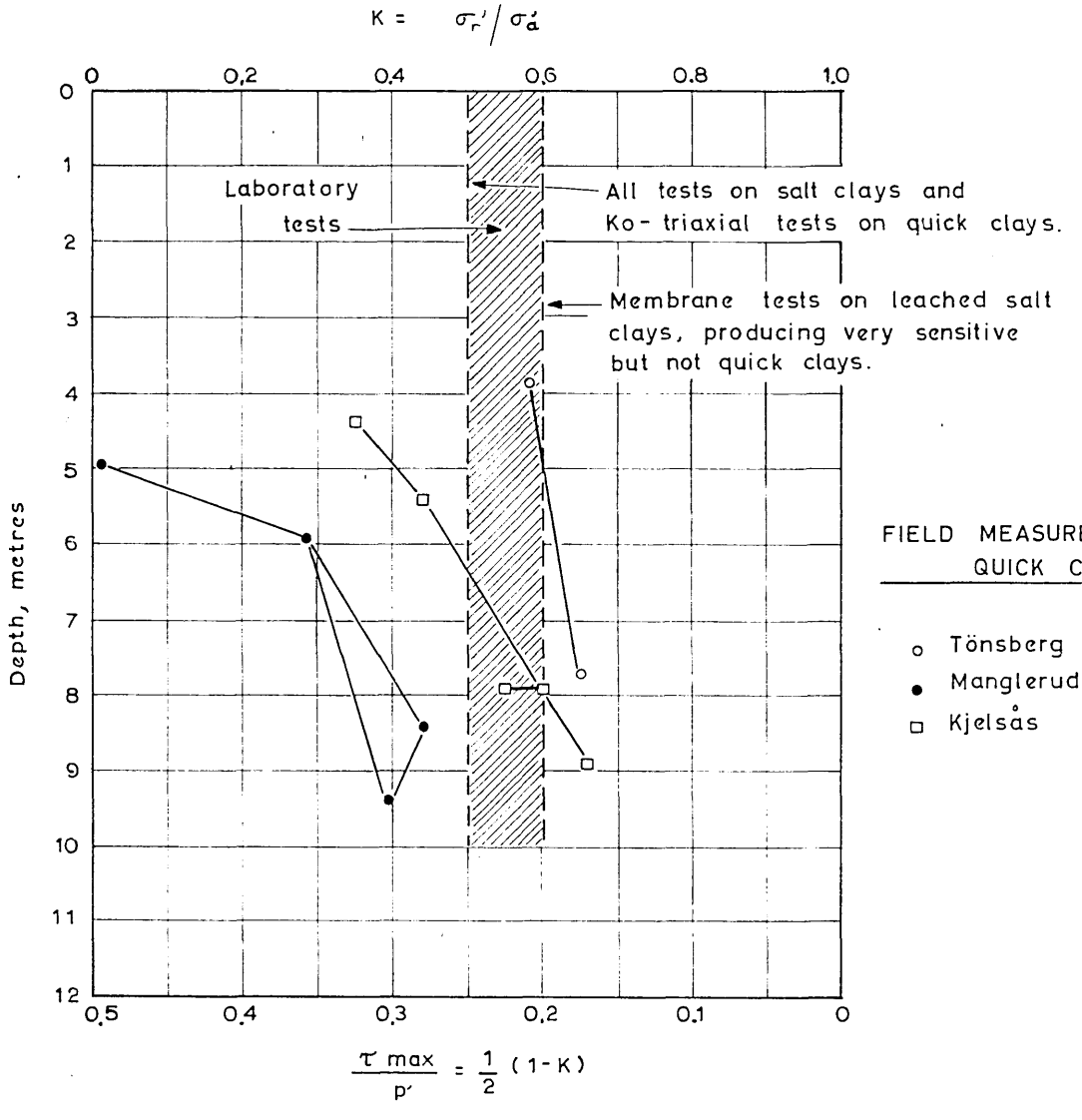
E denotes earth-pressure gauge
W denotes water-pressure gauge







IN SITU STRESS MEASUREMENTS AT KJELSÅS



SUMMARY OF Ko - MEASUREMENTS IN THE LABORATORY AND K - MEASUREMENTS IN THE FIELD

Chapter 3. Investigations of Natural-Slope Failures in Soft Clays.

1. Introduction.

The second method that was used to investigate the shear resistance of quick clay was to make a study of slope failures in quick clays. The purpose of the study was not to determine the safety factor of the slope at the time of failure based on measured shear strengths in the field or in the laboratory, but rather to consider each slope failure as a large scale test and to approximately determine from stability calculations the effective-stress shear-strength parameters of the clay. For most slides there is generally information lacking on topography and ground-water pressures, and this is true for the slope failures that will be dealt with in this thesis, but it was thought that such a stability study would provide a reasonable picture of the behaviour of the clays and also give some indications of the magnitudes of the shear-strength parameters.

Very early in the study three decisions were made.

1. Most slope failures in sensitive clays were compound failures; that is, following an initial failure other slides occurred in succession. For purposes of making stability calculations, both topography and water pressures are required if effective-stress analyses are to be done, and such information is only available for the conditions prior to the initial slide. Hence, only initial slides were studied.

2. Even when in possession of information concerning topography and water pressure, a stability analysis for the purpose of determining shear-strength parameters is of no value unless the approximate size and location of the initial slide is known, because the magnitudes of estimated strength parameters are dependent on the location of the assumed slip surface. This point will be dealt with in more detail later. Therefore, only those slides were studied for which information was available concerning the location of the initial slide.

3. The original plan was to make a stability study of slides in quick clay, but it was soon found that few such cases had been investigated and for

those that had the initial slides were located not only in quick clay but also to an important extent in weathered clay. Therefore, the study was broadened to include slides in any soft clay.

In the following sections of this chapter a description will be given of the method used to estimate strength parameters from stability calculations, a number of case histories of slope failures will be presented, and finally a synthesis will be presented of the results of the calculations and conclusions will be drawn. The locations of the slide sites are given in Fig. 3-1.

2. Method of Stability Calculation.

In most effective-stress stability calculations, shear-strength parameters of the soil are first assumed and a number of slip surfaces are investigated until a minimum safety factor is obtained. Such a method based on minimum safety factors has the disadvantages that (i) the location of the critical slip surface is dependent on the magnitude of the shear-strength parameters assumed for the soil and (ii) it is necessary that shear-strength parameters for the soil be assumed (or measured) before a stability analysis can be started.

A slightly different approach has been taken to the problem for this study. Instead of dealing with safety factors, similar calculations are made to determine the average stress conditions (shear and normal effective stresses) along a number of assumed slip surfaces and from these results to determine the most critical average stresses and the locations of the most critical potential slip surfaces in the slope. If failure has occurred in the slope and some information is available concerning the location of the slip surface, the average stresses on this slip surface can be determined and can be used to estimate approximately the shear-strength parameters of the soil.

Consider the slope drawn at the top of Fig. 3-2. The average shear stress $\bar{\tau}$ along the circular-arc slip surface 1 can be calculated as $\bar{\tau}_1 = \Sigma w . \sin\alpha / \text{arc length}$, and this value can be drawn as a horizontal line in a $\bar{\tau}\,\bar{\sigma}_n'$ plot. Next assume that the strength parameters of the soil are equal

to any two specific values, say c' and tg φ', and calculate F_1 against failure by such means as Bishop's method of slices (Bishop, 1955). (Alternatively, assumptions could be made of F_1 and tg φ' / F_1, and c' / F_1 is obtained directly from the calculations without using trial and error procedures that are necessary if F_1 is the unknown). The average mobilized shear-strength parameters along slip surface 1 are c' / F_1 and tg φ' / F_1. These values can be drawn in the $\bar{\tau}\,\bar{\sigma}_n'$ diagram and the intersection at Point 1 of this line and the $\bar{\tau}_1$ line previously drawn defines the average normal effective stress $\bar{\sigma}_{n\,1}'$ along the slip surface. The stress point $\bar{\tau}_1\,\bar{\sigma}_{n\,1}$ for slip surface 1 defines the average stress condition along the slip surface and is for practical purposes independent of the assumed values of c' and tg φ' used in the calculations. In fact, it is seen from the $\bar{\tau}\,\bar{\sigma}_n'$ diagram that an infinite number of c' tg φ' combinations between the limits of c' = 0, tg φ' = tg φ'_1 and c' = $\bar{\tau}_1$, tg φ' = 0 could be used to satisfy the average stress condition of $\bar{\tau}_1\,\bar{\sigma}_{n\,1}'$ on slip surface 1. A check on any calculation can be made by assuming new values of c' and tg φ', calculating the safety factor and the mobilized shear-strength parameters and finally plotting the line in the $\bar{\tau}\,\bar{\sigma}_n'$ diagram. If the calculations are correct all lines should pass through Point 1. In similar manner, other slip surfaces can be studied, and for each slip surface a $\bar{\tau}\,\bar{\sigma}_n'$ point is obtained. An enveloping curve is drawn to these points and this curve represents the most critical average stress conditions on slip surfaces passing through the slope. The accuracy of the enveloping curve is improved by considering larger numbers of slip surfaces.

In this thesis only circular-arc slip surfaces will be considered, but the method described above can be easily adapted for generalized slip surfaces by modifying the methods of stability calculation used for such surfaces (see Janbu, Bjerrum and Kjærnsli, 1956; Janbu, 1957; Kenney, 1956). The method can also be modified to deal with cases in which there are two soil types, one having known strength characteristics (usually granular materials) and the other having unknown strength characteristics.

An example of the use of the method is given in Fig. 3-2. The stability of the given slope was first calculated by the usual safety-factor method for two assumed sets of strength parameters. The results of these calculations are given as contours of safety factor, and for each case the slip

surface giving the lowest safety factor is shown. It is apparent that the location of the critical slip surface is dependent on the shear strength parameters (strictly speaking, only when c' > 0).

For each slip surface considered in these calculations, a $\overline{\tau}\,\overline{\sigma}_n'$ point was then determined and plotted in a $\overline{\tau}\,\overline{\sigma}_n'$ diagram, dots being used for Case 1 surfaces and crosses being used for Case 2 surfaces. The critical slip surfaces for the two cases are plotted as circles and are numbered 1 and 2. An envelope has been drawn to include all the points for both cases and it is the critical-stress curve for the slope. It is seen that this curve fits all the data, independent of whether the slip surfaces were used for the purpose of studying Case 1 or Case 2.

If we now reconsider the problem of studying the stability of this slope, it is apparent that the critical-stress curve can be obtained from calculations of a small number of slip surfaces. If the strength parameters of the soil are known, a corresponding strength line can be drawn and the minimum safety factor determined by finding the minimum ratio between shear strength of the soil and average shear stress along slip surfaces, as given by the critical-stress curve. A graphical method is used in Fig. 3-2. For the shear-strength parameters used in Cases 1 and 2, the same safety factors and the same locations for the critical slip-surfaces are obtained by this method as by the more common safety-factor method.

In this chapter an attempt is made to estimate the shear-strength parameters of the soil involved in slope failures. Such an estimate is impossible unless there exists some information concerning the location of the slip surface of the initial slide. In practice, this is done by examining a number of slip surfaces and calculating the critical-stress curve for each slope. If the location of the actual slip surface is approximately known, stability calculations can be made of slip surfaces covering a region around the actual slip surface, and in this way the part of the critical-stress can be located that represents approximately the average stress conditions in the slope where failure occurred. In Fig. 3-2, for example, if the slide was known to have been shallow and near the brook, similar to slip surface 1 and not similar to slip surface 2, a portion of the lower part of the critical-stress curve would be chosen and this would give the result that the soil had a small c' -value and

a large tg φ'-value, in contrast to the result obtained if it had been known that the initial slide was large and deep. The accuracy with which c' and tg φ' can be determined in practice is, however, largely dependent on the accuracy of the stability analyses for small slips; that is, the accuracy of the tangent to the critical-stress curve is dependent largely on the accuracy of the points defining the curve in the small-stress region.. Of most importance in this respect is the influence of soil discontinuities, such as the surface weathered zone. Such a strong zone can have considerable influence on the stability of small surfaces, and can lead to apparently large c'-values that are measures of the influence of the strong surface zone rather than being properties of the softer soils underlaying the weathered soils.

In the following sections of this chapter, this type of shear-strength study will be made of a number of slope failures in soft clays. Where possible data will be presented on laboratory strength tests, in particular the effective stresses at failure, and these stresses will be compared to the estimated average failure stresses in the slope. This method of comparing stresses at failure, rather than shear-strength parameters, directs attention to the magnitudes of stress found in the field and therefore the magnitudes that should be used in the laboratory. The results of the stability analysis of the slope in Fig. 3-2 clearly indicate that the average value of σ_n' for the critical slip surface for Case 1 is less than 2 t/m^2 or 0.2 kg/cm^2, and very few people, including the author of this thesis, have previously performed many tests at such a low stress level.

Four important Scandinavian slides will not be considered in this chapter, and they are Surte, Sweden (Jakobson, 1952), Guntorp, Sweden (Fellenius, 1954), Bekkelaget, Norway (Eide and Bjerrum, 1954), and Furre, Norway (Hutchinson, 1961). Each of these slides occurred very rapidly and involved large areas. In the cases of Surte and Furre there were witnessed initial local slides that spread, and in the cases of Guntorp and Bekkelaget there existed steep banks rising from flat terrain that might have failed initially and caused the subsequent movement of large areas. The fact of the matter is that the development mechanism of these slides is not known and therefore stability calculations are of little value. The behaviour of the Furre slide will be considered in Chapter 5.

3. Selnes Slide.

General. The Selnes slide is so called because it occurred at Selnes wholly within the farm of J. Selnes. It has been described briefly by Rolfsen (1966).

Selnes is located in the valley of the Namsen River, 150 km north of Trondheim. In the photographs in Fig. 3-3 it is shown that the landscape consists of large bedrock outcrops and flat post-glacial marine-clay terraces into which streams have eroded their valleys to depths of 10 or more metres. The clays at Selnes are soft and extremely sensitive ($s_u \approx 2$ t/m^2, St > 100). Details concerning site topography are given in diagram A and this information was obtained from a map made from aerial photographs taken in 1960, five years before the Selnes slide; Photo 1 is one of these photographs. The accuracy of the contours is $\pm$ 0.5 metre. In 1959 a small slide occurred at the bend of the stream and can be seen in Photo 1, the slide masses nearly blocking the stream. This material was subsequently eroded away. It is also seen in Photo 1 that the banks of the stream upstream from the 1959 slide were steep and prone to surficial slides originating from stream erosion. It was in this area that the 1965 slide began.

The slide occurred at about 3:00 P.M. on Easter Sunday, April 18th, 1965. The following information gives a clear picture of the slide sequence.

1. Three weeks before the slide occurred, two boys on skis noticed a crack in the snow 5 to 8 cm wide and extending 20 metres along the upper edge of the stream bank, and upon returning home one of the boys remarked to his family that soon there would be a slide at Selnes. It must be pointed out that small clay slides along streams in this part of Norway are common occurrences and most farms experience them. This would explain the keen observation and the uncanny prediction of the boy. However, it is unlikely that the boy thought in terms of anything other than a small slide.

2. The owner of the farm located in the lower left corner of Photo 2 was in the front room of his house when he heard a noise and he looked out to see that a part of land upstream from the 1959 slide had moved out and that the slide debris was flowing down the stream valley in the direct-

-ion of his farm. The slide developed very rapidly as piece by piece of land slid down from all sides of a growing crater. Instability progressed downstream towards building A and backwards towards the farm. The witness emphasized that the slide developed very rapidly and estimated that it was only a few minutes before buildings B, C, D and E were taken by the slide.

3. On the Selnes farm, the slide was first noticed by people in building F who saw the slide progressing piece-by-piece backwards towards the barn B. A woman rushed to warn the people in building D, and at the same time as she reached the building buildings B and C completely collapsed, and buildings D and E were taken moments later. The direction of slide development then abruptly changed towards the south and it was some time later that building F slid out.

The conditions after the slide are seen in Photo 2. The disturbed clay, that had the consistency of a viscous fluid, flowed about 400 metres down the stream valley and became dammed against a growth of small trees at a sharp bend of the stream. The slide masses also flowed upstream, damming the stream and caused a lake to form. The buildings B and C, that were originally attached, became separated during the slide, and whereas part B was carried well down the stream valley, part C became caught in an eddy current of remoulded quick clay and remained near the original position of building A. This eddy current was observed by J. Selnes, who remained in building D, when the flowing clay carried building D over the original position of building A and then down the stream valley. Contours of the surface separating undisturbed clay and slide masses (diagram A) indicate that a gully was eroded along the northeastern part of the slide boundary. The facts that (i) building C remained in an eddy current to one side of the flow of quick clay and (ii) building F and the trees, that were taken in the very last phases of the slide, were located along the centreline of the gully, seem to indicate that the main flow of clay from the western part of the slide area was concentrated along the northern boundary, presumably because this location provided the best direction by which the flowing clay could by-pass the piled-up debris that blocked the upper part of the stream.

Location of the Slip Surface. Soundings and vane borings were made to determine the areal extent of the slide in those areas covered by slide masses, and also to locate within this slide area the boundary between intact undisturbed clay and slide masses. In principle the method was to determine the location of the deepest major discontinuity in the soil-strength profile at many points, this discontinuity being the upper boundary of undisturbed clay and therefore the slip surface.

(a) Soundings. The original ground surface was silty and compact and it could not be penetrated with a pointed steel sounding rod. Similarly, the streambed was covered with cobbles and gravel and this material could not be penetrated; it was easily identified as being streambed and was easily distinguished as not being silty top soil. On the other hand, the undisturbed quick clay, underlaying the surface soils and the streambed materials, gave no resistance to penetration of the sounding rod. This was also the case with disturbed quick clay in the slide masses. Therefore, by sounding through the slide debris it was possible to identify the streambed materials and the original surface soils. Where the elevations of these soils corresponded to the pre-slide elevations given in diagram A, it could be concluded that these areas had not been disturbed by the slide and therefore were located outside the actual slide. If the elevations were greater than the pre-slide elevations, this meant that the sounding rod had encountered a flake of surface soil, but no conclusions could be drawn regarding the presence or absence of the original ground surface. On the other hand, if a sounding gave no resistance to penetration, it meant that the soils encountered were remoulded or undisturbed quick clays without a surface-soil covering, and therefore this sounding was located within the actual slide area.

(b) Vane borings. To locate the boundary between undisturbed quick clay and the overlying slide debris borings were made with a hand vane (Andresen and Sollie, 1966) to depths of 9 metres. Borings were first made in an area at the back of the slide where the undisturbed clay was exposed and it was found that the strength of undisturbed clay was between 1.5 and 3.0 t/m^2 and that of remoulded clay was about zero. Measured values above or below 1.5 to 3.0 t/m^2 would indicate that the soil being

tested was not undisturbed quick clay. Large strength values gradually decreasing with depth to become equal to those for undisturbed quick clay would indicate an undisturbed profile of weathered clay over non-weathered clay. Such a profile was not found within the slide boundaries and in all borings the undisturbed soil was found to be quick clay. Perhaps the most dependable evidence of the presence of undisturbed quick clay was its very brittle behaviour. Up until failure the vane turned only slightly, but as soon as the clay's strength had been reached the vane rotated very suddenly. In contrast, the remoulded clays and the insensitive weathered soils in the slide masses all exhibited a plastic-type failure. Vane tests were made at vertical intervals of 0.25 metres and therefore the slip-surface location at each boring has an accuracy of about $\pm$ 0.25 metres.

The locations of the soundings and vane borings are given in diagram B together with the elevations of the slip surface. In both diagrams B and A are given the contours of the surface of undisturbed clay (slip surface), and the boundary between the slide area and the undisturbed but covered area. Typical sections have been given in diagram C.

The first point of interest concerning these results is the bowl-shaped depression in the slip surface found at the bend of the stream. This depression is also clearly shown in Section 1. It is reasonable to conclude that it is the scar of the initial slide. The fact that it is located where the stream was actively eroding its bank would imply that stream erosion had been an important factor contributing to instability. It is also important that the slip surface did not extend much below the level of the streambed; that is, it was shallow.

The second point of interest is that outside of this bowl-shaped depression the slip surface is very regular and in many places the slope of the slip surface in the direction towards the stream is linear. That it is so regular in spite of the fact that the slide was seen to develop retrogressively as a series of very small slides, seems to indicate that any major irregularities were smoothed away by the eroding action of the moving slide masses. It also indicates that plane debris-covered slip surfaces do not necessarily indicate flake-type slides.

The third point of interest is the elongated depression along the north side of the slide area. The boundaries of this depression are very sharp. From the shape of this depression in cross section (Sections 4, 5 and 6), from the fact that the wreckage of building C marks the location of an eddy current to one side of the depression, and from the fact that the location of building F and the trees mark the direction of flow of the slide masses, it can be concluded that this depression was caused primarily by erosion by the slide masses coming out from the western part of the slide area. At the deepest point the gully is about 4 metres below the pre-slide streambed. Such erosive powers of moving clay indicate that slip surfaces can be greatly modified if the sliding clay is forced to pass between two obstructions.

Stability Calculations of the Initial Slide. Stability calculations were made of the slope shown in Fig. 3-4, and this cross section is typical for the slope in which the initial slide occurred. Given in the cross section are the locations of determined points on the slip surface. These points do not necessarily locate the slip surface of the initial slide, but they do provide the lowest limit of its location. Therefore, when choosing slip surfaces for the stability calculations of the initial slide, most attention was paid to surfaces passing through or above the contact between slide debris and undisturbed clay.

Water pressures were not measured in the field and therefore it was necessary to assume them. Two distributions were used in the calculations. The first is shown in Fig. 3-4 and gives slight overpressures at the toe of the slope. Such a condition has always been found by the Norwegian Geotechnical Institute in field investigations of clay slopes bordering a stream. The ground-water surface has been drawn on the basis of information gained from the farmer; (i) on the terrace the ground-water surface was very close to the ground surface and shallow tile drains had to be used in order to farm the area, and (ii) the slope was always very wet and small surficial slides were frequently occurring. The second water-pressure distribution that was assumed is not drawn in Fig. 3-4 and was simply that the water pressures everywhere increased hydrostatically with depth below the ground-water level. The first assumed distribution is believed to be the most reasonable. The second assumption, without doubt, underestimates the water pressures near the foot

of the slope and therefore in stability calculations it will lead to overestimates of normal effective stress on the slip surfaces.

The locations of 7 slip surfaces are given in Fig. 3-4, together with a $\bar{\tau}\,\bar{\sigma}_n'$ diagram for each of the two assumed water-pressure distributions. Slip surfaces 6 and 7 extended below the contact between slide masses and undisturbed clay, and therefore they are not relevant to the study of the initial slide; they only help to define the critical-stress curve for assumed larger slides. From the admittedly weak information concerning cracks at the top of the slope, and from the location of the boundary between slide masses and undisturbed quick clay, it is thought that the portion of the critical-stress curves between points 4 and 6 best represents the average stress conditions at the time of the initial slide.

Estimation of Shear-Strength Parameters for Quick Clay. The critical-stress curves were used to estimate the shear-strength parameters of the Selnes quick clay, but two corrections were necessary; a correction for the influence of the bowl-shape of the initial slide, and corrections for the strengthening influence of the surface zone of weathered soil. It has been shown that a stability study of the centre section of a bowl-shaped slide will lead to an overestimation of the mobilized shear strength by approximately 5% (Sevaldson, 1956; Kenney, 1956; Kjærnsli and Simons, 1962), and therefore in Fig. 3-4.B the critical-stress curves have been lowered by $0.05\,\bar{\tau}$.

A correction for the weathered clay involved a determination of the fraction of slip surface that passed through weathered clay and an estimation of the strength difference between weathered and unweathered clays. The results of the vane borings showed clearly that, along the edge of the stream at the foot of the slope, quick clay existed within 1.5 metres of the ground surface. At the top of the slope there was a weathered zone some 3 to 5 metres thick, and for short slip surfaces (for example, surface 1) this weathered clay would influence about 20% of the slip surface below the tension crack, and for larger slip surfaces there would be an influence over about 15% of the length of the slip surface. The strength difference between weathered and unweathered clay is primarily in the c' parameter and this difference was estimated from two small slides that had occurred in the

vicinity of the Selnes farm in the same terrace on which the farm was located. Here, the depth of weathering below the terrace surface is about 4 to 5 metres, and because the slides occurred along a stream valley that was no more than 3 metres below the terrace surface, it follows that these slides were wholly within weathered clay. To obtain a maximum c' - value for the weathered clay it was assumed that the pore pressures were large and that the effective stresses were equal to zero. Stability calculations gave $c' \approx 1\ t/m^2$, and the value is larger than that measured in the laboratory for weathered soft soils at other sites in Norway. By assuming that the difference between weathered and unweathered clays corresponded to $c' \approx 1\ t/m^2$ and by using the percentage of slip surface in weathered soil that was eliminated previously, the average shear stresses in the quick clay found by stability calculations were reduced by $\Delta\bar{\tau} = 0.20 \times 1 \approx 0.20\ t/m^2$ for the small slip surfaces and by $\Delta\bar{\tau} = 0.15 \times 1 \approx 0.15\ t/m^2$ for the larger slip surfaces.

The corrected critical-stress curves are given in Fig. 3-4, diagram B and represent the average stresses along slip surfaces in quick clay at the time of the initial slide. Those portions of the curves that are believed to represent the conditions for the initial slide (between the points for slip surfaces 4 and 6) are shown as heavy lines and tangents to these curves give the following approximate shear-strength parameters.

Hydrostatic water pressures: $c' \approx 0.2\ t/m^2$, $\text{tg}\,\varphi' \approx 0.50$

Overpressures at foot of slope: $c' \approx 0.4\ t/m^2$, $\text{tg}\,\varphi' \approx 0.50$

$c' \approx 0.6\ t/m^2$, $\text{tg}\,\varphi' \approx 0.42$

For $c' = 0$, the maximum values of $\text{tg}\,\varphi'$ were obtained by drawing lines from the origin to the furthest end of each critical-stress curve.

Hydrostatic water pressure: $c' = o$, $\text{tg}\,\varphi'_{min} \approx 0.55$

Overpressures at foot of slope: $c' = 0$, $\text{tg}\,\varphi'_{min} \approx 0.56$

Comparisons of Shear Strengths. Samples were taken of the quick clay from 0.5 to 1.5 metres below the exposed slip surface shown in the upper left corner of Fig. 3-3.B. At this level the clay was 7 metres below the original ground surface and had the following characteristics.

$w \approx 34\%$, $w_L \approx 22\%$, $w_p \approx 17\%$

$s_u \approx 1.6\ t/m^2$, $St > 100$

Particles $< 2\mu \approx 40\%$, Salt content ≈ 0.6 gm/l

Drained triaxial compression tests on isotropically consolidated samples were performed at stresses that covered the range of average stresses obtained from the stability calculations. Volume changes during consolidation were about 2% and from water-content measurements and from previous K_o experience it was known that most of this water came from the thin surface zone of disturbed clay surrounding the samples. Some test results are presented in Fig. 3-4.C.

Three additional tests were performed on samples of this clay that had been opened to the air and then stored in this condition for six months at 100% relative humidity. The clay weathered and when tested it had the following characteristics.

$w = 35\%$, $w_L = 35\%$, $w_p = 27\%$

$s_u = 1.8\ t/m^2$, $St = 7$

It can be seen from diagram C that the weathered samples behaved similarly to the unweathered quick clay. The tests gave the following shear-strength parameters.

For $\sigma_n' = 2$ to $6\ t/m^2$, $c' \approx 0.5\ t/m^2$, $\text{tg}\,\varphi' \approx 0.52$

For $\sigma_n' > 6\ t/m^2$, $c' \approx 0$, $\text{tg}\,\varphi' \approx 0.60$

The laboratory results have been transferred to diagram B and they are in good agreement with the estimated average stresses in the quick clay at the time of the initial slide. The agreement corresponds to $F = 1.05$ for overpressures at the foot of the slope, and $F = 1.20$ for hydrostatic water pressures.

Too little was known concerning the undrained strengths of the weathered clays to make total-stress stability calculations. However, from Fig. 3-4.A it is found that the average shear stresses along slip surfaces were about 2 t/m^2, a value similar to the undrained strength of the quick clay, but probably much smaller than the value for the weathered clay. Therefore it would be expected that for shallow slip surfaces such as those in Fig. 3-4.A, total-stress stability calculations would give a minimum safety factor greater than unity; that is $F > 1$.

4. Ullensaker Slide.

General. The Ullensaker slide was first described by Bjerrum (1955) and since that time it has been used many times as a typical example of a quick-clay slide. Photographs taken before and after the slide are presented in Fig. 3-5. After the successful investigation of the initial slide at Selnes, it was decided to return to Ullensaker to attempt an investigation of its initial slide.

The general topography of the area is given in Fig. 3-5 and was obtained from aerial photographs taken three months before the slide occurred. The area consisted of a flat marine-clay terrace into which small brooks have eroded gullies. At the site of the slide there was a very small brook at elevation 156 metres, about 12 metres below terrace level.

The slide began on the morning of December 23rd, 1953. The owner of the farm on which the slide occurred had been working in the carpentry building until 9:30 A.M. when he left it to go to his house to eat. He returned at 10:00 A.M. to find that the slope had failed and that the back edge of the slide boundary was just at the edge of his carpentry building. The slide scarp was steep and about 4 to 5 metres high. He described the slide scar as resembling in shape a sand pit, having a flat bottom and steep sides. Two tall trees that had grown on the farm side of the brook had been moved directly across the brook and were tilted back towards the farm. One-half hour later another small slide took away half of the carpentry building. No further movements and no cracks

were observed that day but the farmer, remembering that he had encountered quick clay when digging his well and that this clay could be excavated with a bucket, decided to move his family off the farm. At about 2:00 A.M. the next morning there was a noise followed by a smaller noise about two hours later. When daylight came it was found that the farm had been taken by a quick-clay slide and that the quick clay had flowed a distance of 1.5 km before being dammed up at a road embankment. The autumn months preceeding the time of the slide had been drier than usual but the summer had been uncommonly wet. There was considerable water flowing in the brook but this was not unusual for that time of year. Although it was nearly Christmas, there was no frost in the ground.

The farmer was able to locate accurately on photographs the position of the slide scar that he first saw, and this information is given in diagram C. The length along the brook was 30 to 35 metres and the upstream boundary corresponds exactly to the final slide boundary. On looking at the shape of the slide scar that was first observed, the width measured up the slope from the brook is as large as the length along the brook and the scar is narrow at the top, an unlikely shape for an initial slide, and it will be assumed that this scar was the product of at least two slides, the initial slide having approximately one half the width of the cavity seen by the farmer, as shown in diagram C.

Another point of interest is the locations of the two large trees, before and after the slide. They moved with the initial slide but were not swept away by the quick clay that flowed out through the gap during the periods of major sliding. This means that their locations in diagram C mark the outer boundary of the flowing quick clay, and by drawing a curve through their positions to join the left-hand side boundary of the slide, the approximate location is found of the area that could have been affected by erosion of flowing quick clay.

Location of the Slip Surface. Vane borings were made to locate the slip surface in the area of the initial slide. This slip surface would not necessarily be the slip surface for the initial slide but would give the lower limit for the initial slide. In the vane borings the slip surface would appear as a major discontinuity in the strength profile of the soil, and to

everyone's surprise, considering the borings were made 13 years after the slide occurred, this discontinuity was clearly evident in most of the borings, the strength profile being erratic above the slip surface and being regular below.

The results of the borings are given in Fig. 3-6.B. Two observations can be made. First, in the borings close to the brook the slip surface was located in stiffer, presumably weathered, clay, whereas in the borings located away from the brook the slip surface was located in soft sensitive clay. This indicated that near the brook the slip surface was shallow and was located in the surface-weathered soil, whereas further back under the slope the slip surface was below the weathered zone. Second, the slip surface has the form of a gully orientated parallel to the original stream. In diagram C, where the contours of the slip surface are superimposed on the original ground contours, it is apparent that considerable soil has been removed from the bank of the brook opposite to the initial slide, presumably by erosion of the slide masses flowing out through the narrow gap. From this evidence it is probable that the slip surface of the initial slide was altered by subsequent erosion and that its location was above the slip surface found from the vane borings and given in diagrams C and D. Borings have not yet (1966) been made in the upstream half of the initial slide that is located outside the main flow of the slide masses and therefore where there was probably less erosion.

Stability Calculations of the Initial Slide. Stability calculations were made of the slope shown in Fig. 3-7. Results of borings and laboratory work performed in 1954 (Stabell and Höy, 1954; Bjerrum, 1955) are also given, and the locations of these borings are shown in Fig. 3-6.C.

The water-pressure distribution within the slope is based on measurements made in 1966 in the slope on the opposite side of the brook along a section located in Fig. 3-6.A. This slope has been assumed to be similar to that which failed, having a slightly smaller inclination and rising to the same terrace level. Near the top of the slope the water pressures increased hydrostatically below the ground-water surface; in the middle of the slope there were small overpressures increasing with depth; and at the foot of the slope the piezometric head measured at a

depth of 3.2 metres was 2.2 metres above the ground surface and at a depth of 6.2 metres the piezometric head was 3.3 metres above the ground surface.

The locations of the slip surfaces that were used in the stability calculations are given in Fig. 3-7. B and the results of the calculations are presented in the $\overline{\tau}\,\overline{\sigma}_n'$ diagram. In these calculations a tension crack 1 metre deep has been assumed but no other allowance has been made of the fact that the surface soils were weathered. The location of the initial slide was probably somewhere between slip surfaces 3 and 6. No correction has been made for the influence of the bowl shape of the slip surface.

Comparison of Shear Strengths. In Fig. 3-7. D are presented the results of drained triaxial compression tests performed on samples obtained from three borings along the investigated section shown in Fig. 3-6. A. One boring was at the brook, another boring was near the slide boundary and the third boring was a short distance up the slope on the other side of the brook. By comparing the index properties of these clays with the 1954 results of tests on samples from the initial slide area, it was determined that the new samples were representative of the clays in the initial-slide area. The results of the triaxial tests indicate that most of the tests (28 of 33 tests) are located within the narrow strength limits of $c' \approx 0$, $\text{tg}\,\varphi' \approx 0.64$ and $c' \approx 0.2\ \text{t/m}^2$, $\text{tg}\,\varphi' \approx 0.70$, and these tests included both weathered clays having $s_u \approx 5$ to 16 t/m^2 and unweathered quick clays having $s_u < 3\ \text{t/m}^2$. The five samples that plot above the upper limit are all from within the surface 2 metres and all had undrained strengths $s_u > 20\ \text{t/m}^2$. These shallow clays would in practice be the first to be affected by the formation of tension cracks and it is questionable that these clays have any stabilizing influence on the slope when the slope is in such a condition that small surface movements are occurring.

The strength envelopes obtained from the triaxial tests are drawn in Fig. 3-7. C and they agree well with the results of the stability calculations.

The undrained shear strength of the soil that was involved in the initial slide can be roughly estimated from the 1954 vane borings and from those made in 1966, and it varies from values larger than 20 t/m^2 near the ground surface to about 1 t/m^2 beneath the brook. The average value would certainly be much greater than the average shear stress along slip surfaces of $\overline{\tau} \approx 1.5\, t/m^2$ obtained from the stability calculations (Fig. 3-7.C), and therefore the safety factor of the slope obtained from a total-stress analysis for slip surfaces similar to the actual slip surface would be much greater than unity; that is, $F > 1$.

5. Göta Slide.

General. The slide at Göta is one of the largest slides to have occurred in Scandinavia in modern times. In area it extended for a distance of 1.5 km along the Göta River and, at its widest point, 250 metres in from the river bank. It destroyed a sulphide factory and caused the deaths of three people. This slide is of particular interest because it occurred in very sensitive clays and because it started as an observed small initial slide and then very rapidly increased in size. It has been described by Odenstad (1958).

Information concerning the site is given in Fig. 3-8. Göta is situated on the Göta River 50 km north of Göteborg and 3 km south of Lilla Edet. At this location the Göta River is bordered by a marine-clay terrace that rises from an elevation of 7.5 metres near the river to an elevation of 9 metres 250 metres from the river. Bedrock outcrops close to the river, and in the area of the initial slide it is located at elevation -20 metres below the river. Sandy and gravelly materials overlaying bedrock were found in the borings.

The undrained shear strength values given in Fig. 3-8 were obtained from two borings (Nos. 1 and 2) made in 1951 and one boring (No. 3) made after the slide; these borings are located in the site plan. The sensitivity of the clay was large (about 20) but no information was given concerning plasticity and water content. At another section 200

metres upstream from section AA three other borings were made through the slide masses. Here the undrained shear strength increased with depth from 2 t/m^2 near the ground surface to 6 t/m^2 at a depth of 30 metres. The clays near the river had sensitivities of about 10 whereas they were much more sensitive 200 metres from the river where they were also closer to the outcropping bedrock. The liquid and plastic limits of the clays were 65% and 30%, and the water contents corresponded to liquidity indices of about 0.75 near the river and 1.3 at a distance 200 metres from the river.

Description of the Slide. On the morning of June 7th, 1957, a crack was noticed 15-25 metres inland from the top of the steep bank just downstream of the factory area. The area was cordoned off and boats docked at the quay were given order to sail. The crack, 130 metres long, was crescent shaped, running essentially parallel to the river. The narrow crack widened only several millimeters during the first hour, but thereafter its speed of development increased steadily until at 11:25 A.M., three hours after its discovery, the initial slide occurred. A thorough investigation of the surrounding area had been made in the meantime but no other cracks had been found.

Witnesses clearly saw the initial slide, but the following rapid development of instability, simultaneous in upstream and inland directions, caused them to run for their lives. The witnesses were in agreement that, following the initial slide, the ground slid out piece after piece, and not as one large movement. The slide continued in this manner and destroyed the factory located at the widest section of the slide. Because there were few surface cracks upstream from this point after the slide, in spite of large horizontal movements, it appears possible that the final 2/3 area of the slide moved as three large flakes, the middle 1/3 of the slide area being one, and the upstream 1/3 of the slide area being comprised of two flakes.

The initial slide reduced the width of the river from 120 metres to about 60 metres and created a wave that travelled rapidly upstream. The crews of two boats on the river estimated the wave height as being 5 metres and 6 to 8 metres, and by the time it had travelled 2.5 km up-

stream to the sluice gates at Lilla Edet it had subsided to 3.5 metres. It was the opinion of Odenstad (1958) that the slide might have developed in the upstream direction at the same speed as the wave, the stability of the river bank being successively disturbed by the low-water trough that followed the wave.

Stability Calculations of the Initial Slide. The slope investigated is given in Fig. 3-8 and is taken from Odenstad's paper; in the paper no mention is made of the accuracy of the topography of this slope. The only information available concerning water pressures in the soil is that over-pressures of unmeasured magnitude existed in the river area in the sand and gravel layer overlaying bedrock. The water-pressure distribution given in Fig. 3-8 was drawn on the following assumptions: (i) near the edge of the slope the ground-water surface is at a depth of 2 metres (about elevation 5 metres) and below this the water pressures increase hydrostatically to the sand and gravel layer, and (ii) the water pressure in the sand and gravel layer below the river (assumed to be located at about elevation -20 metres) corresponds to a pressure level at elevation 5 metres, and that the water pressures decrease linearly in the vertical direction towards the river bottom.

Stability calculations for seven slip surfaces were made and the results are presented in a $\bar{\tau}\bar{\sigma}_n'$ diagram. The surfaces were located so as to extend from the general area of where the initial crack was seen to different locations below the river. Because the location of the actual slip surface is not known, no comment will be made concerning the shear-strength parameters of this soil.

Comparison of shear strengths. No data are available on laboratory triaxial tests on clay from the Göta slide. However, Simons (1964) has tested clay from a site near Lilla Edet that is only several kilometres from the initial slide. It had the following index properties (see also Bjerrum and Wu, 1960):

$w \approx 66\%$, $w_L \approx 59\%$, $w_p \approx 29\%$

$St \approx 15$

In drained tests at stresses very much larger than those given in Fig. 3-8.D, the shear-strength parameters were $c' \approx 0$, $tg\varphi' \approx 0.64$. This strength line has been drawn in Fig. 3-8.D and compares favourably with the estimated in situ stresses at failure.

The undrained shear strengths measured in the vicinity of the initial slide were generally larger than 3 t/m^2. In comparison with the $\bar{\tau}$-values in Fig. 3-8.D that range from 1.5 to 2.5 t/m^2 for likely slip surfaces, the measured strengths are large, giving safety factors at the time of failure of about F = 1.5 to 2. Odenstad (1958) made a comprehensive total-stress stability investigation and he obtained F = 1.6.

6. Naddum Slide.

General. The Naddum slide (Simons, 1961) occurred in the bank of the Höland River that has eroded its bed into a plain of soft quick clay. The river bank that failed was only 5 metres high. The surface clays are weathered to a depth of 3 to 4 metres, and they are stiff near the ground surface, as shown in the photograph in Fig. 3-9.

A few days prior to the slide cracks were noticed in the road at the end of the slide nearest the house, and on September 1st, 1958, a few minutes before the slide occurred, a lorry driver approaching from the south saw movements in the road and reversed his truck out of danger. The slide started at the northern end and progressed in stages rapidly southward (total time was less than a minute). The width of the slide was limited by steeply-sloping bedrock that is visible in the photograph. The river level at the time of the slide was elevation 117.8 metres.

The properties of the soils are indicated in Fig. 3-9.B. Below the weathered soil the clay is soft and extremely sensitive. At approximately elevation 112 metres there is a continuous silt layer into which five piezometers were installed, and the results of measurements over a one-year period showed that the water pressures in the silt layer corresponded to a piezometric level at elevation 121 metres and that they were essentially independent of the water level in the river. The water pressures drawn in diagram C correspond to approximately linear variation of water pressure

from the silt layer to the ground-water surface or to the bottom of the river.

Results of Stability Calculations. The results of stability calculations are given in diagram D and it is seen that the estimated values of $\bar{\sigma}_n'$ are very small. The position of the actual slip surface is probably between slip surfaces 3 and 6, and this portion of the critical-stress curve in diagram D has been drawn as a wide line.

The critical-stress curve in the quick clay was approximately determined with the following method. It was assumed that the difference between the strengths of the weathered and unweathered clays corresponded to $\Delta c' \approx 1\ t/m^2$, and if the weathered clay affected about 0.2 of the length of the slip surfaces (this is true of slip surfaces 3 to 6), then the critical-stress curve for the quick clay would be located below the curve obtained from the stability calculations by the amount $\Delta\bar{\tau} = 0.2 \times 1 = 0.2\ t/m^2$, as shown in Fig. 3-9.D.

Comparison of Shear Strengths.

In Fig. 3-11.D are given the results of a number of undrained triaxial compression tests performed on samples of quick clay from this site but primarily from a nearby site of similar but slightly more plastic clay; there was no apparent difference in the test results for the two sites. No tests were performed on weathered soils, and no drained tests were performed. The extrapolated results compare favourably with the estimated in situ stresses at failure, bearing in mind that certain objections can be raised concerning the use of undrained tests for determining shear-strength parameters of soft clays (such as, that the effective stresses decrease throughout most of the tests' duration).

Simons (1961) has made total-stress stability calculations based on the results of the vane borings. The result was a safety factor $F = 1.05$. Considering that four of the five vane borings were made within the slide area, the reliability of these vane results are in question. If more weight is placed on the results of boring 5 outside the slide area or on the results obtained from another boring in the general vicinity (Simons, 1961), a

safety factor greater than 2 can be obtained, but without more reliable strength measurements at the site of the slide it cannot be said that this value is more accurate than Simons' result.

7. Drammen Slide.

General. The Drammen slide of 1955 has been described in detail by Kjærnsli and Simons (1962). It occurred as a single slip in normally-consolidated insensitive clay.

Information is given in Fig. 3-10. The slide occurred in the north bank of the Drammen River at 10:00 A.M. on the 6th January, 1955, without warning and apparently without any signs of impending instability. Approximately 4000 m^3 of soil over a length of 35 to 40 metres and up to 10 metres in from the river's edge slid into the river, leaving behind an almost vertical face. A small amount of fill had been dumped prior to the slide and the water level in the river at the time of the slide was 1 metre below normal.

In Fig. 3-10 are given 4 slope cross sections. Sections A and C are stable sections at either end of the slide, and Section B is through the centre of the slide. Section D is a stable part of the river bank downstream from the slide. Many borings were made at the site and, because they all gave much the same result, only two borings near Section B are presented. The natural clay is uniform with depth and exhibits sensitivities equal to about 8. Granular fill had been dumped on top of the clay in some places, and piles had been driven into the slope, as indicated in Fig. 3-10.

Stability Calculations. An extensive program of water-pressure measurement was made in both the slide area and in the stable areas. It was found that the water pressures increased hydrostatically with depth below the ground-water surface level and that no overpressures existed in the clay.

In those sections where there was fill above the clay, it was assumed that the fill had the following shear-strength parameters: $c' \approx 0$,

$\text{tg}\,\varphi' \approx 0.64$.

Summaries of the results of the stability calculations are given in the $\bar{\tau}\,\bar{\sigma}_n'$ diagram. For each section the critical-stress curve is given together with the locations of the stress points for slip surfaces that are believed to cover the range of locations of the most likely slip surfaces. Section B, through the centre of the slip, gives the highest critical-stress curve. Of the stable sections, Section D gives the highest curve if the piling is assumed to have no stabilizing effect on the slope. If consideration is given only to slip surfaces passing below the piling (surface D'), then Sections D, A and C have approximately the same critical-stress curves.

Comparison of Shear Strength.. Many drained and undrained triaxial compression tests were performed, all of them at stresses greater than those at which failure occurred in the field, but some of them approaching the field stresses. Very consistent results were obtained from both types of test, giving the following shear-strength parameters: $c' \approx 0$ to $0.4\ t/m^2$, $\text{tg}\,\varphi' \approx 0.64$. These results are plotted in the $\bar{\tau}\,\bar{\sigma}_n'$ diagram and compare well with the results of the stability claculations.

Measurements of undrained shear strength showed that this strength increased with depth, being equal to approximately $1\ t/m^2$ within the depth of clay where the slip occurred. This value is appreciably less than the average shear stress of $\bar{\tau} \approx 1.5$ to $2\ t/m^2$ found from the stability calculations. Kjærnsli and Simons (1962) made total-stress stability calculations on a large number of slip surfaces and obtained the following results:

Section A, $F = 0.71$
Section B, $F = 0.47$
Section C, $F = 0.58$
Section D, $F = 0.71$

It is perhaps of significance that at this site, where $F < 1$ for total-stress stability analysis, there is no surface weathered zone. This zone greatly increased the average undrained shear strengths at the other slide sites

for which F > 1 was found.

. Lodalen Slide.

General. The Lodalen slide has been described in detail by Sevaldson (1956). It occurred in 1954 at a railway marshalling yard constructed in the valley of the Lo River. Thirty years previous the river had been redirected through a tunnel in bedrock, the riverbed filled, and the slopes of the valley steepened from their natural inclination of 1:2.5 to 1:2. In 1949 the area was further expanded and 2.5 metres of the slope was excavated but the inclination remained unchanged at 1:2. During the last few years prior to the slide three sheds and some filling materials had been placed at the top of the slope but these were not affected by the 1954 slide and therefore could not have contributed much to the instability of the slope.

The slide occurred between 5 and 6 A.M. on October 6th, 1954, involving approximately 10×10^3 m^3 of soil. The length of the slide was 50 metres along the slope and the width was 40 metres in from the foot of the slope. There were no witnesses.

Typical soil data are given in Fig. 3-11. The sensitivity values ranged from 3 to 15 and the equivalent NaCl content in the pore fluid ranged from 3 to 15 gm/l. From investigations of the micro-fossils in the clay it was determined that these fossils were not in chronological order with depth in the borings, older clays being found above or between younger clays, and this meant that the area had previously experienced slides or had been covered with slide masses. This would explain the somewhat variable nature of the clays.

Stability Calculations. The water-pressure distribution given in Fig. 3-11 is the same as that used by Sevaldson and was determined by him from measurements at four locations and at four depths at each location. There are only very small overpressures at the foot of the slope, the reason being that the filled-in Lo riverbed is located only 30

metres from foot of the slope and provides a drainage level at about elevation 3 metres, that is 6 metres below the ground level at the foot of the slope.

Results of the stability calculations are given in the $\overline{\tau}\,\overline{\sigma}_n'$ diagram. From the location of the slide boundary at the top of the slide and from two locations of the slip surface found from examination of samples from borings, it seems that the actual slip surface is similar to slip surface 4, and for this reason the portion of the critical-stress curve at point 4 is believed to represent the average stress conditions at the time of failure.

Comparison of Shear Strengths. A considerable number of triaxial compression tests were performed on tube samples and on block samples and the results of those performed at the lower stresses are given in Fig. 3-11 (NGI records). There is a large scatter of the results, possibly because of large property variations of the clays, but also possibly due to piston friction in the testing apparatus that was used at that time (1955). The average of these results is in good agreement with the results of the stability calculations given in the $\overline{\tau}\,\overline{\sigma}_n'$ diagram.

Sevaldson made total-stress stability calculations of many slip surfaces using measured undrained shear strengths from a number of borings. The minimum safety factor that was found was F = 0.93.

9. Conclusions.

1. The main purpose of the slope-failure study was to attempt to determine from stability calculations the effective-stress shear-strength properties of quick clay. This was accomplished in the case of the Selnes slide, and to lesser extents for the Göta and Naddum slides, all of which gave similar results. For the Selnes quick clay, even after taking account of the shape of the slip surface and the influence of the surface weathered soil in order to obtain the smallest strength values, the determined strength characteristics corresponded to $\overline{s}/\overline{\sigma}_n' \approx 0.57$ to 0.62, and by drawing tangents to the adjusted critical-stress curves the following shear strength parameters were obtained: $c' \approx 0.2$ to 0.5 t/m^2, $\text{tg}\,\varphi' \approx 0.46$ to 0.50; these values have no fundamental significance and simply describe

tangents to specific portions of a curve. These strength characteristics were in agreement with the results of laboratory drained compression tests on quick-clay samples from Selnes and Ullensaker. The laboratory stresses were purposely kept at about the same magnitude as those existing in the field, and the samples showed little tendency towards volume compression during shear.

The quick clays that are involved in the initial slide of a natural-slope failure are in an overconsolidated state as the result of erosion, and therefore the strength characteristics that are obtained from a study of slope failures are for overconsolidated clays. Similarly, if laboratory tests are performed on samples taken from a natural slope and the tests are made at stresses approximately equal to those in the field, the soil in the laboratory will be in an overconsolidated state, comparable to that in situ. For these conditions, reasonably large effective-stress shear strength characteristics were found for quick clay and good agreement was obtained between field and laboratory behaviour.

From this limited experience it appears that the stability of natural slopes in quick clay can be estimated on the basis of standard drained triaxial compression tests, for the important condition that the stresses during testing are about the same in magnitude to those existing in the slope. If, in such cases, large shear strengths are obtained for small volume changes during the test (indicating a stable structure of the clay), it is the author's opinion that these strength characteristics are safe to use in design, even though they might be much larger than those commonly associated with quick clay. On the other hand, if during the laboratory tests there occurs large volume decreases (water expulsion), and if it can be established that these volume decreases are not the result of sample disturbance prior to testing, this behaviour indicates an unstable clay structure at working stresses in the field, and this possibly changes the design problem from being a drained condition to being potentially an undrained condition. If, in the field, the rate of water-pressure dissipation by drainage could be smaller than the rate of water-pressure build-up due to shear strain, there is the possibility of

undrained failure occurring, and this problem is considered in more detail in Chapter 5.

2. Of considerable importance was also the finding that in initial slides of natural-slope failures (for which the water pressures in the soils immediately prior to failure correspond to steady-seepage conditions), quick clays and insensitive clays both exhibited similar effective-stress strength behaviours. This is shown in Fig. 3-12 in which the critical-stress curves for the six slopes have been drawn. The portions of the curves that represent probable failure stresses fall within narrow limits in spite of the fact that the soils vary widely in both sensitivity and undrained shear strength. Similar results were obtained from the laboratory test series on Ullensaker samples, these samples varying within wide limits of sensitivity and undrained strength. Below a depth of 2 metres all the samples exhibited about the same shear-strength parameters for laboratory stresses approximately equal to those in the field, and all these samples exhibited stable structures and small volume decreases during shear. Above a depth of 2 metres the soils were stronger; they have been influenced by dessication and probably by precipitation of cementing agents. It is also true that in the field these soils fail in tension for small slope movements and therefore probably make no contribution to the stability of a slope that is close to failure.

From these limited observations, it is concluded tentatively that the stability of a natural slope in soft Norwegian clays can be estimated approximately using the minimum shear-strength characteristic $s/\sigma_n' = 0.62$. This is the minimum resistance envelope obtained from Fig. 3-12; it includes the influence of the weathered as well as the unweathered clays in the soil profile, and it includes the stabilizing influence of the bowl shape of the slip surface.

3. For most of the slides studied there was inadequate information concerning undrained shear strength of the soils to make comprehensive total-stress stability calculations. From the little data that has been presented it appears that the total-stress method can give both good, but more frequently poor, results and this is in agreement with the findings

of Bjerrum and Kjærnsli (1957).

4. Common features of all the slides studied here, and in fact all witnessed slides in soft clay, are that (i) they began as rather small initial slides, (ii) the size of the initial slides were not necessarily dictated by the total heights of the slopes, and (iii) the final slide areas were in no way dependent on the size of the initial slide (Table 3-1). Also, in those cases for which the location of the initial slide surface is known (Selnes, Ullensaker and Lodalen in this thesis; Vormsund, at present being investigated by the Norwegian Geotechnical Institute; Nicolet, Bilodeau, 1957; Vibstad, Hutchinson, 1965), the slip surfaces were shallow and at their deepest points did not extend more than 2 to 3 metres below the level of the foot of the slope. Thus, from this limited evidence, it is tentatively concluded that in soft clays the problem of slope stability is a local one, and that if small slides can be prevented, then large slides will not occur.

5. Another common feature of all natural slopes in which or near which slope failures have occurred is that water overpressures exist at the base of the slopes. These overpressures are frequently large and reduction of them by drainage trenches or drainage holes would increase the potential shear resistance of the soils significantly at the foot of the slope.

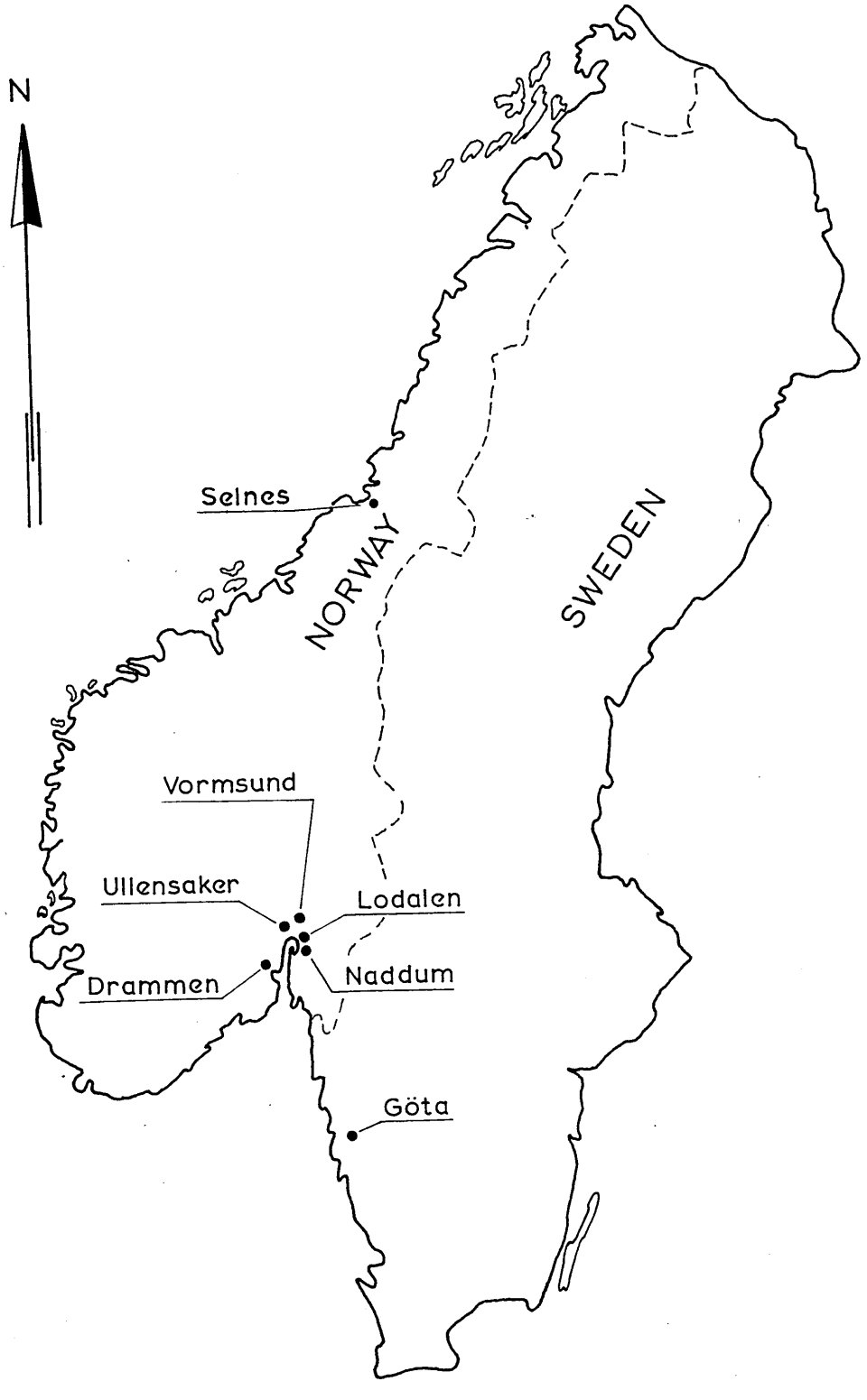
6. Once an initial slide has occurred, and if there exist unsupported steep faces of soil around the slide scarp, there is danger of instability spreading. If the unsupported steep faces are in weak clay (in the undrained sense) these faces will immediately fail, and if the soil is very sensitive the slide masses will slide away leaving other faces to fail. If the soil is both weak and very sensitive, this process can proceed extremely rapidly, as witnessed at the Selnes and Göta slides. If the soil is stiff (dilatant), a steep face might remain stable for some time before failure occurs. Thus, at Ullensaker where the first slides involved primarily weathered clay the rate of retrogression was small, but once quick clay became exposed the slide developed very rapidly. There are two slides in Norway with which the author is familiar where the surface

soils were stiff to depths of 5 to 10 metres and the underlaying soils were weak. In both cases the slides developed retrogressively by failure of the steep rear slide scarp, one (Vormsund) at the average rate of 50 metres per day for 3 days and the other (Ask) at the average rate of 2 metres per day for 30 days.

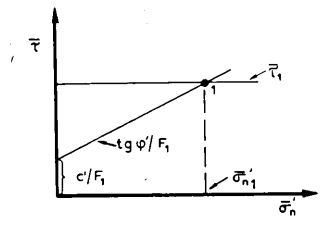
Concerning the rates at which slides involving quick clays can develop, there is absolutely no evidence (with the possible exception of Furre that will be considered in a later chapter) that they liquify in a manner similar to the well-known behaviour of loose sands in certain laboratory demonstrations. In fact, a brief review of available information on flow slides in sands indicated that these flow slides are also not the result of sudden liquifaction over large areas, but rather that they are the result of retrogressive instability similar in mechanism to the piece-by-piece development of quick-clay slides (Koppejan, van Wamelen and Weinberg, 1948; Senour and Turnbull, 1948). Similarities of behaviour of loose sands and quick clays are also apparent after examination of the photograph presented by Jordan (1949) of an earthflow in Florida that resembled in every respect the slide in quick clay at Ullensaker; that is, a shallow bowl-shaped depression 300 metres in diameter and 15 metres in depth, located in a terrace and opening through a narrow gap to a stream valley eroded in the terrace. The main difference between the two slides was that Ullensaker was quick clay and the Florida earthflow was partly indurated clayey sand of Miocene age.

Table 3-1. Comparison of sizes of initial and total slides.

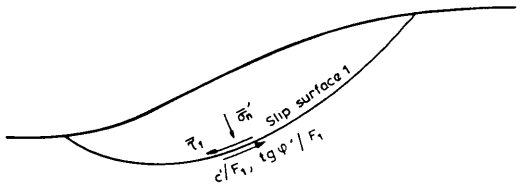
Slide	Clay	Initial slide			Total slide		Total height of slope m
		Width from foot m	Length along foot m	Volume 10^3 m^3	Area 10^3 m^2	Volume 10^3 m^3	
Selnes	Quick	17	30	2.5	18	140	12
Ullensaker	Quick	< 30	30	2	20	160	12
Göta	Quick	30	130	25	150	2000	10^+
Naddum	Quick	11	30	1.5	1.8	11	5
Drammen	Sensitive	18	40	4	little change		15
Lodalen	Insensitive	40	50	10	little change		18

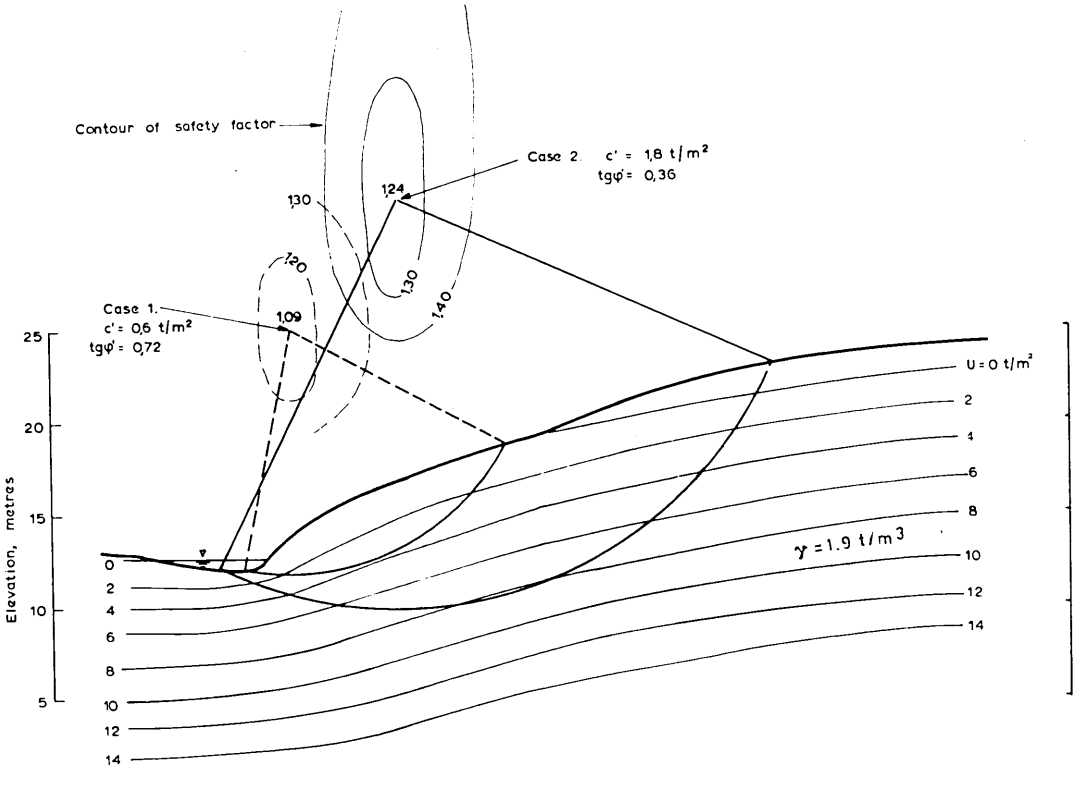


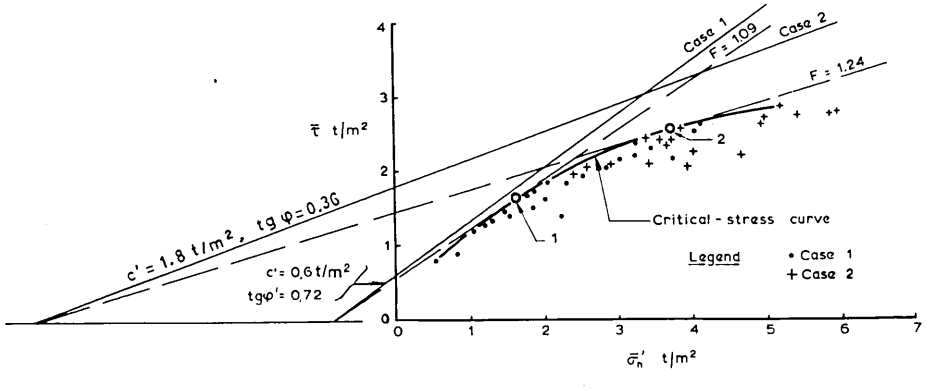
SITES OF SLOPE FAILURES



Average stresses on slip surface 1.







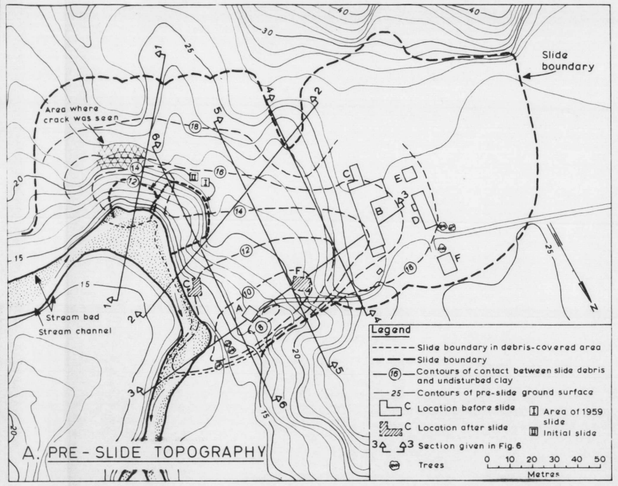
METHOD OF STABILITY CALCULATION

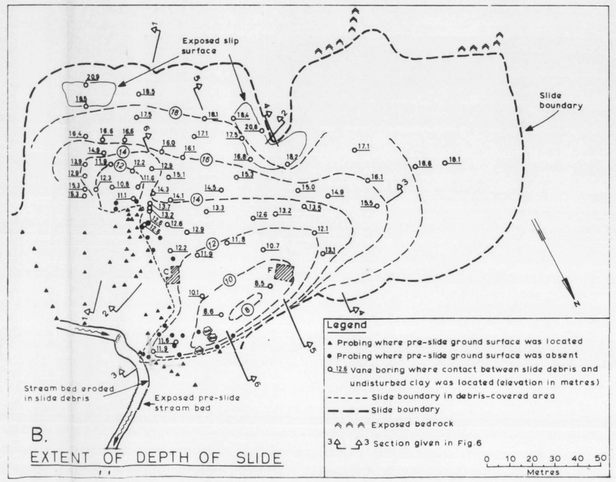


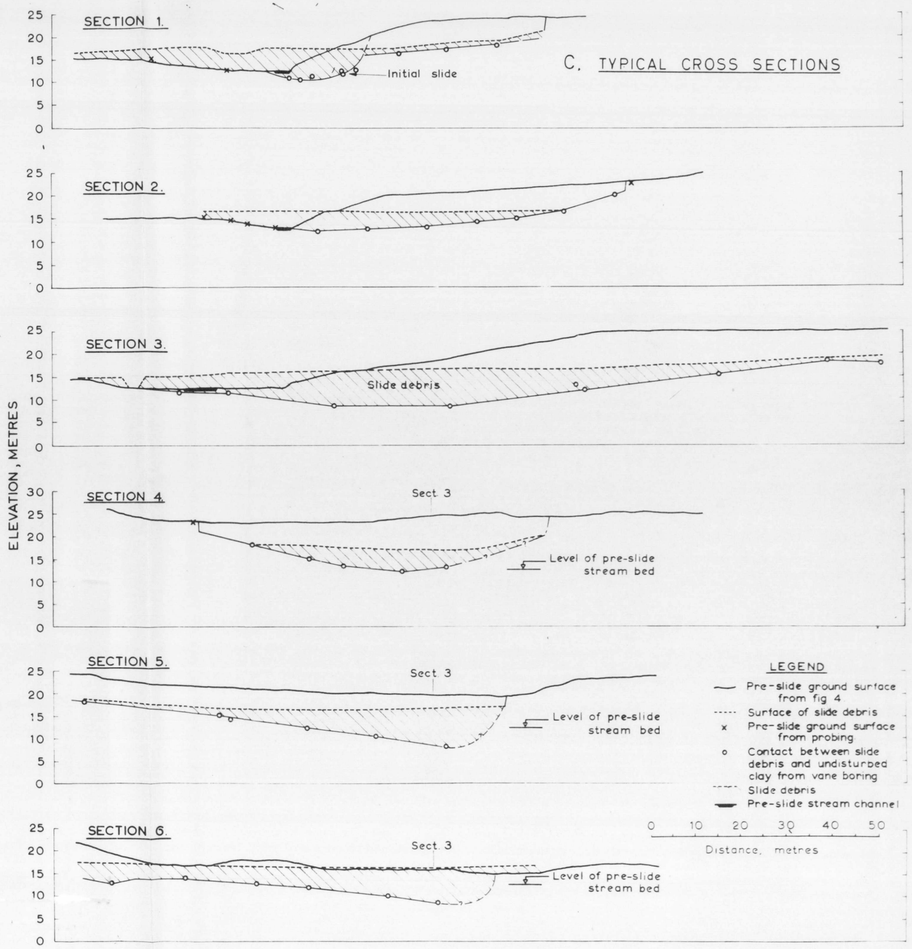
Photo 1. Selnes, 1960



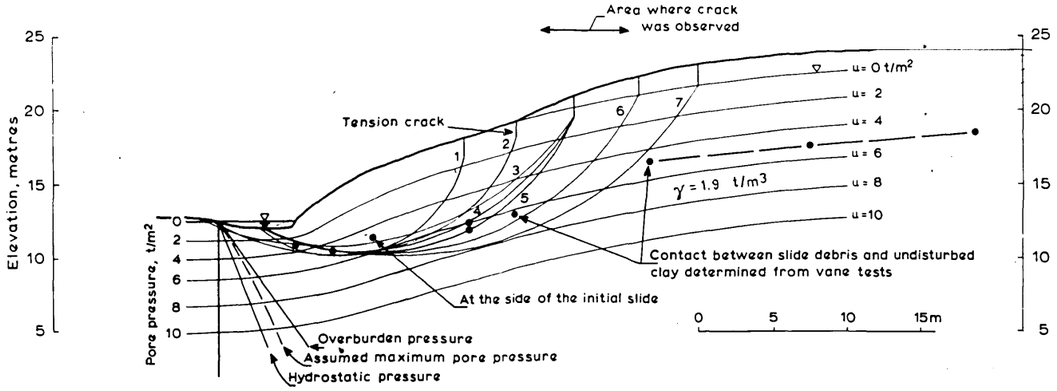
Photo 2. Selnes, 1965



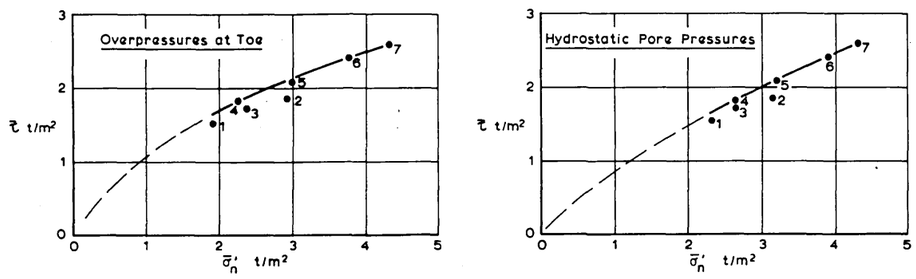




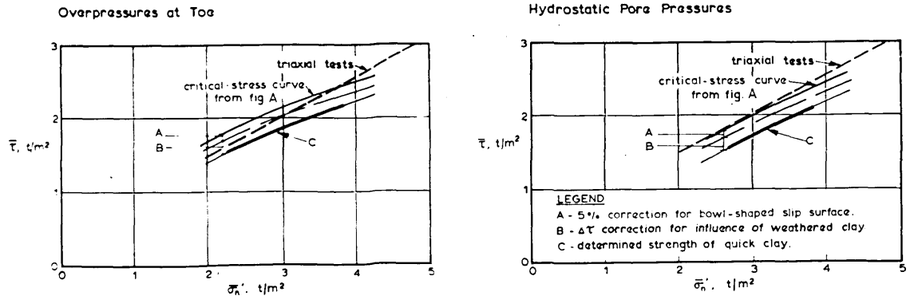
SELNES SLIDE. SITE CONDITONS
BEFORE AND AFTER THE SLIDE



Average stress conditions

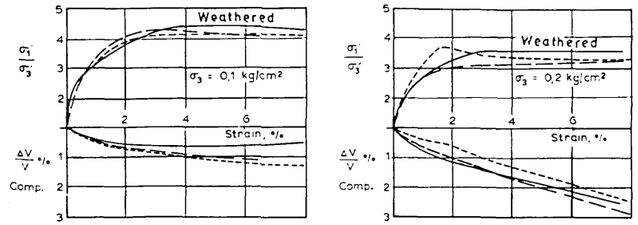


A. RESULTS OF STABILITY CALCULATIONS

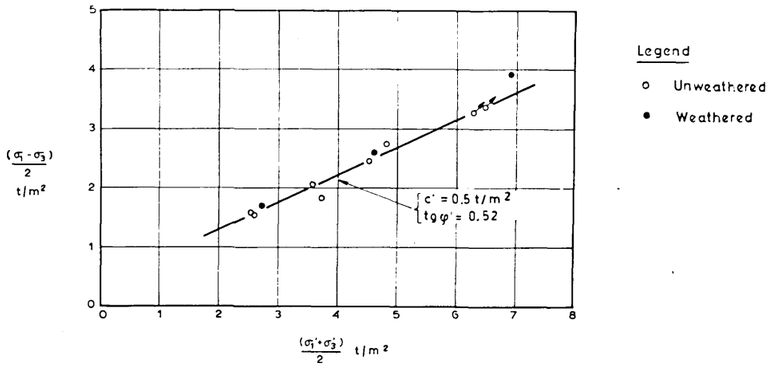


B. SHEAR STRENGTH OF QUICK CLAY FROM STABILITY CALCULATIONS

Typical results of drained tests.



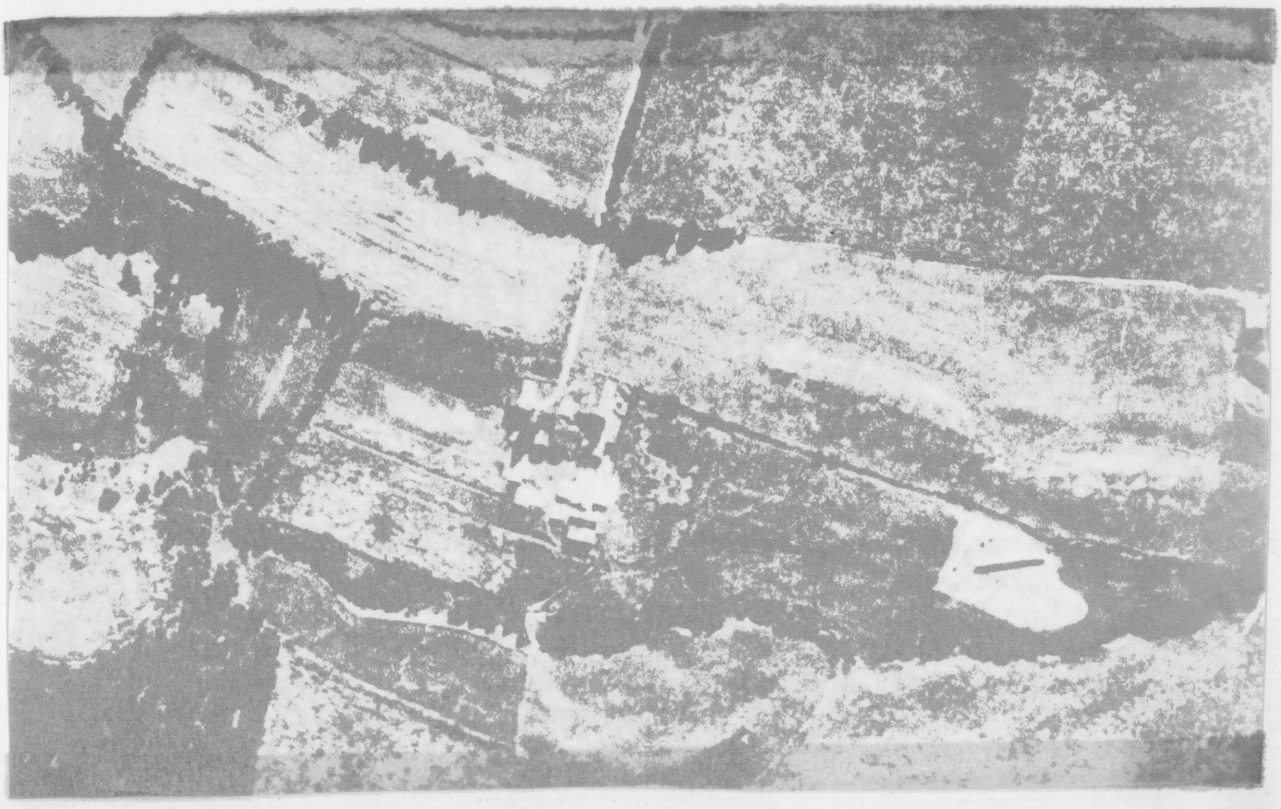
Failure stresses from all tests.



C. SHEAR STRENGTH OF QUICK CLAY FROM TRIAXIAL TESTS.

SELNES SLIDE. SHEAR STRENGTH OF SELNES QUICK CLAY DETERMINED FROM STABILITY CALCULATIONS OF THE INITIAL SLIDE AND LABORATORY TESTS.

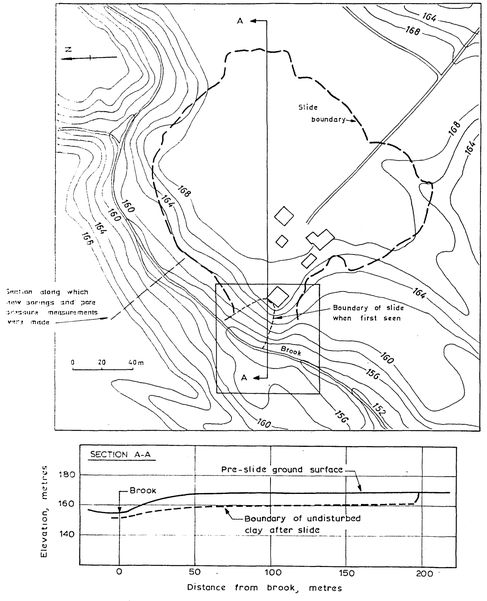
Fig. 3-5



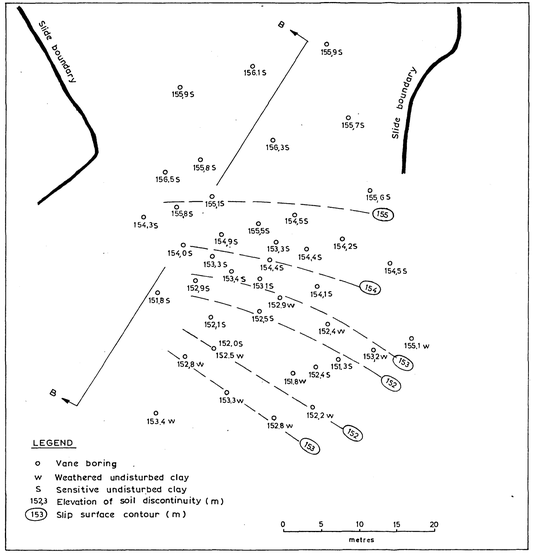
Ullensaker, September 1953



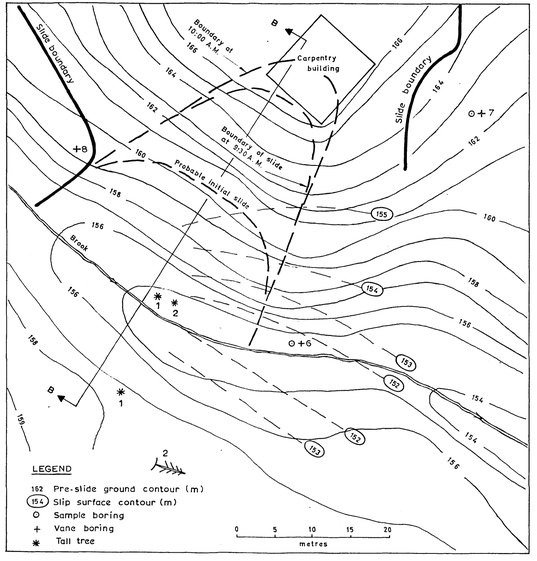
Ullensaker, May 1955



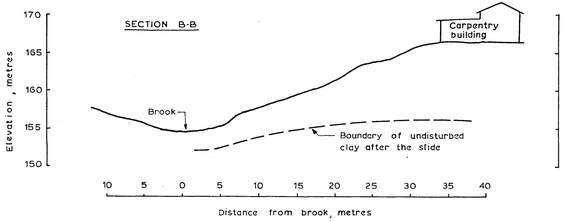
A. Site plan and section through slide crater



B. Results of borings in the area of the initial slide.



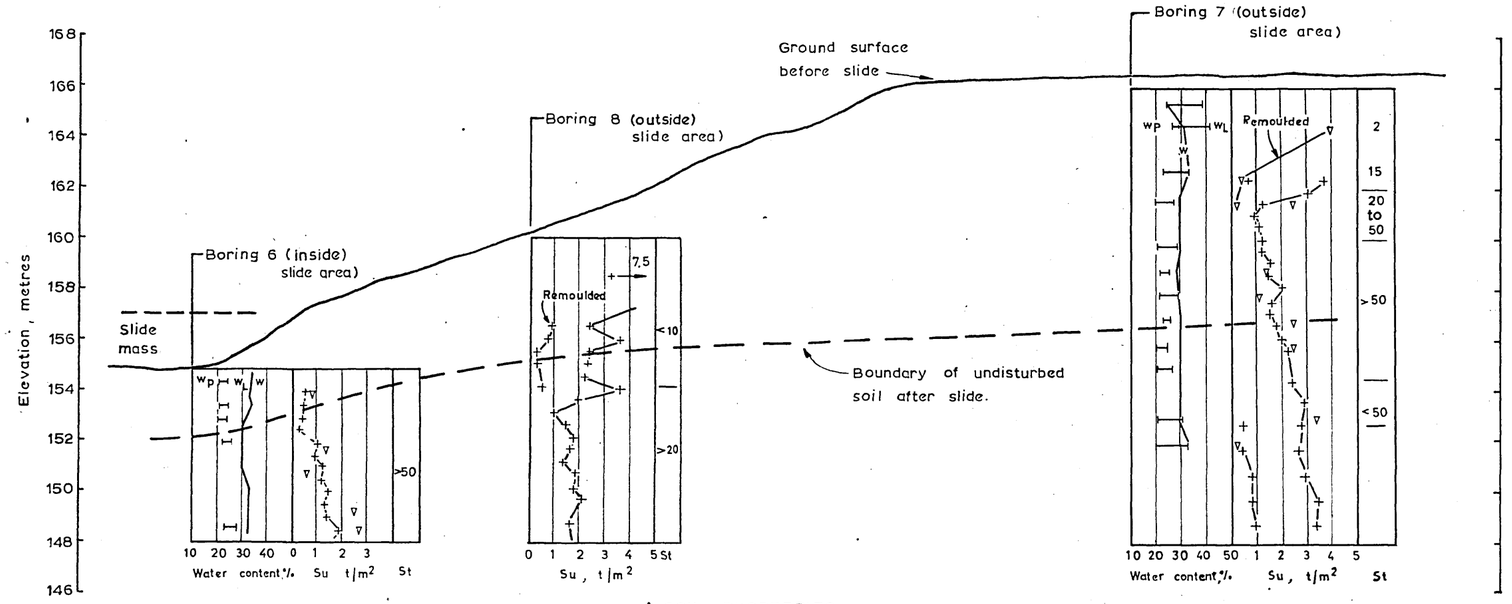
C. Pre-slide topography, location of the initial slide and final contours of slip surface.



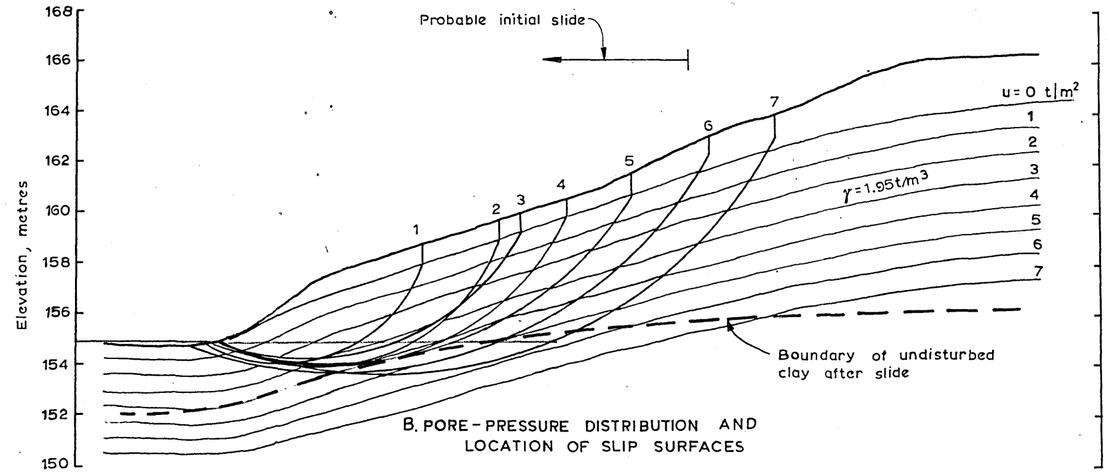
D. Cross section of the initial slide area

ULLENSAKER SLIDE

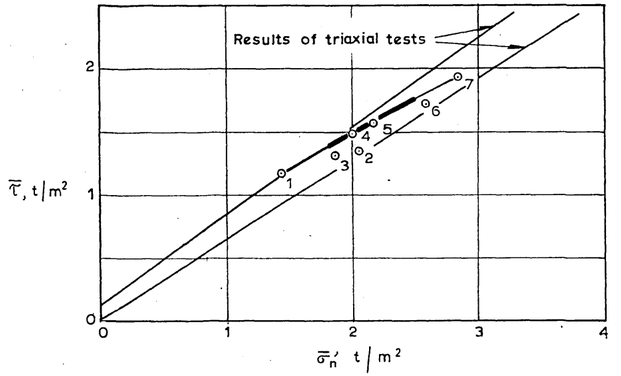
GENERAL SITE CONDITIONS AND RESULTS OF BORINGS IN THE AREA OF THE INITIAL SLIDE.



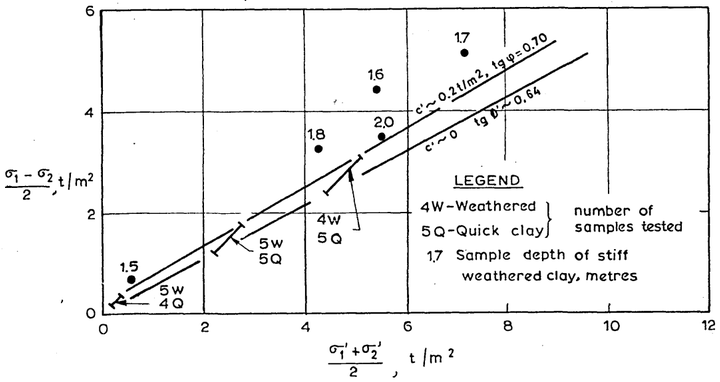
A. SOIL PROPERTIES



B. PORE-PRESSURE DISTRIBUTION AND LOCATION OF SLIP SURFACES

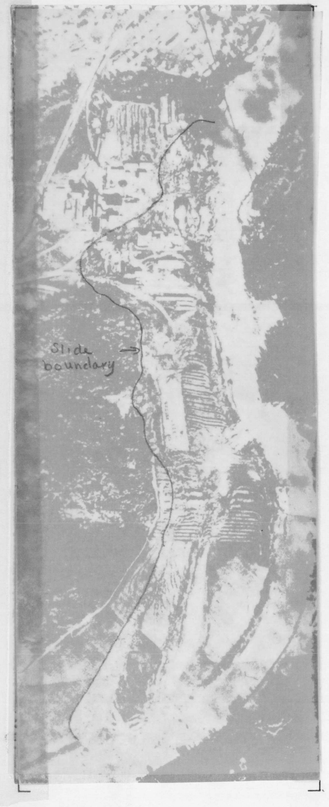


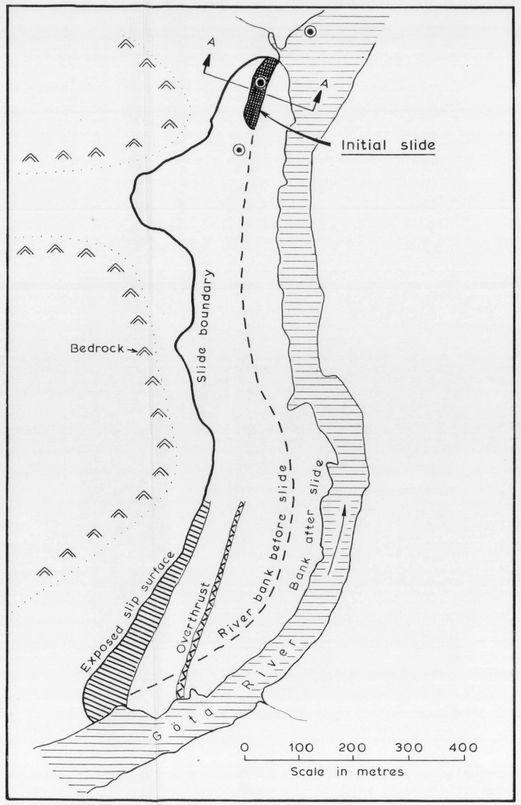
C. RESULTS OF STABILITY CALCULATIONS



D. RESULTS OF DRAINED TRIAXIAL COMPRESSION TESTS

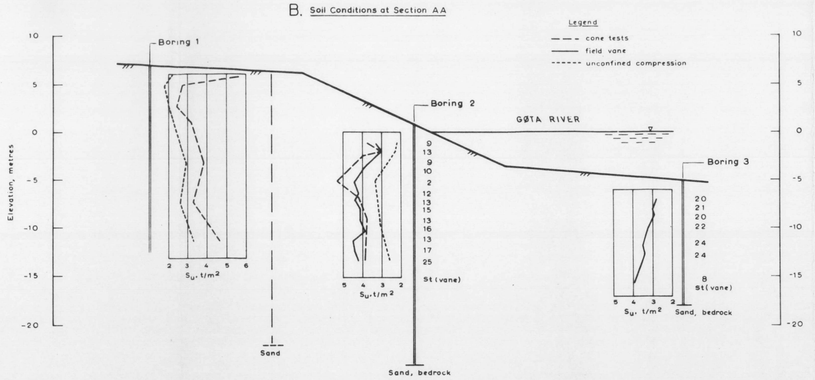
ULLENSAKER SLIDE. SITE CONDITIONS, SOIL PROPERTIES AND RESULTS OF STABILITY CALCULATIONS.



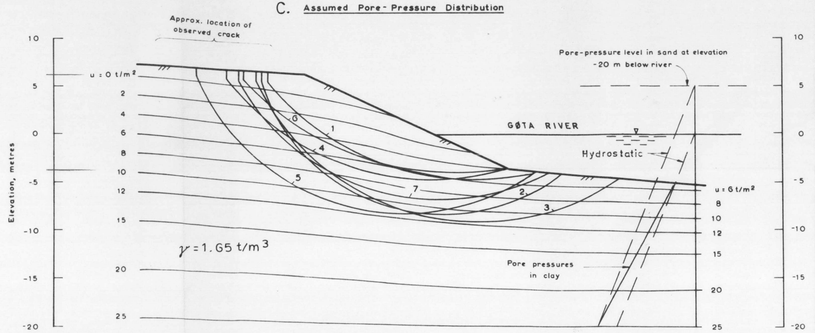


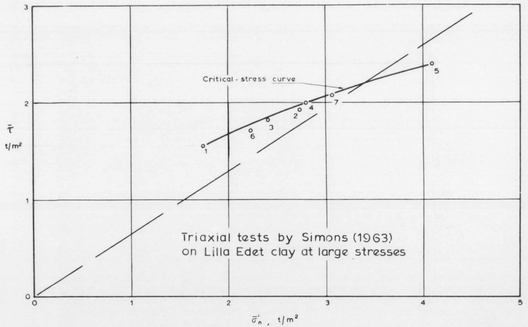
A. SITE PLAN

B. Soil Conditions at Section AA



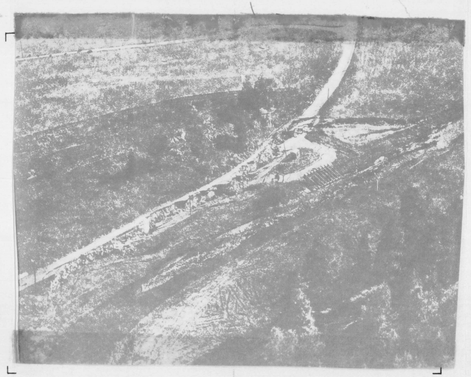
C. Assumed Pore-Pressure Distribution

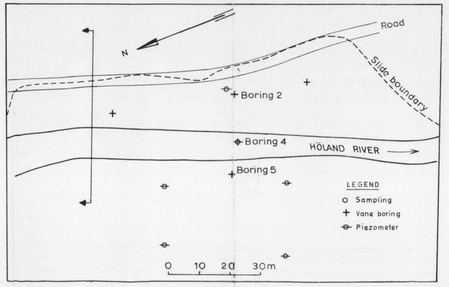




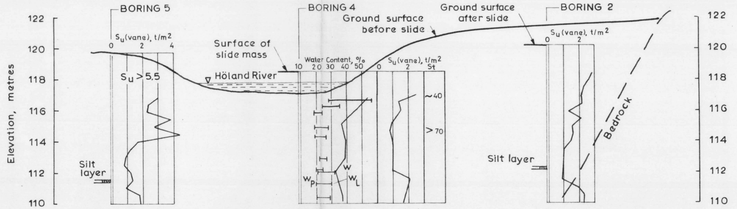
D. Results of stability calculations

GÖTA, INITIAL SLIDE. SITE CONDITIONS, SOIL PROPERTIES AND STABILITY CALCULATIONS.

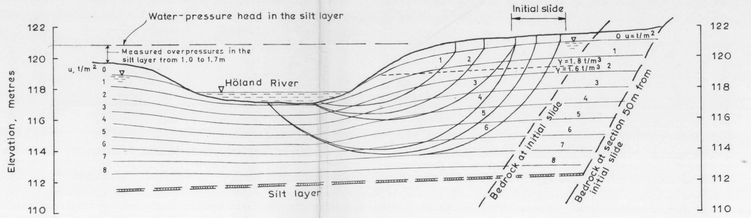




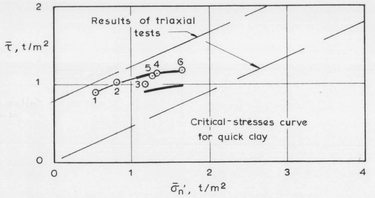
A. SITE PLAN



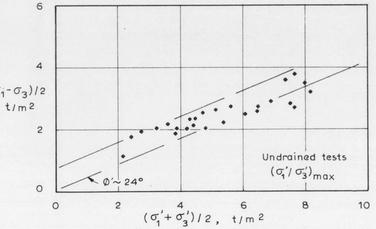
B. SOIL PROPERTIES



C. PORE-PRESSURE DISTRIBUTION AND LOCATION OF SLIP SURFACES



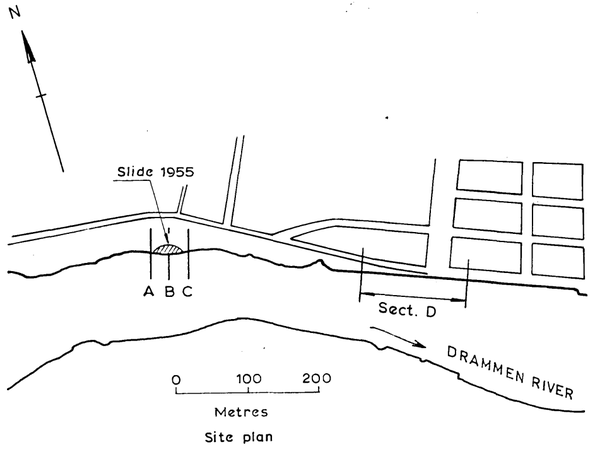
D. Results of Stability Calculations.



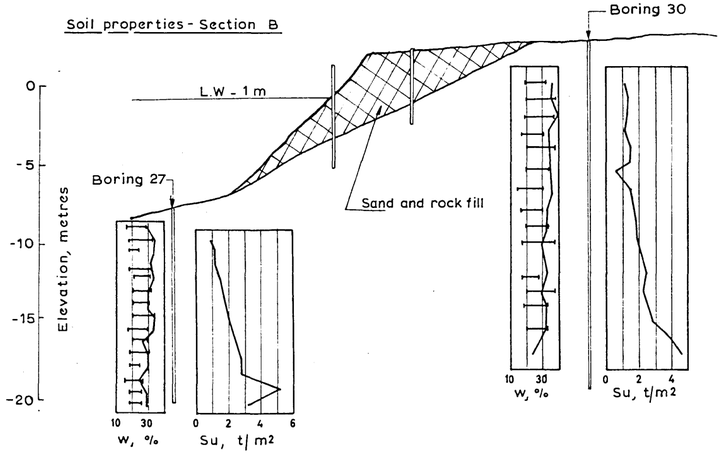
E. Results of Triaxial Compression Tests.

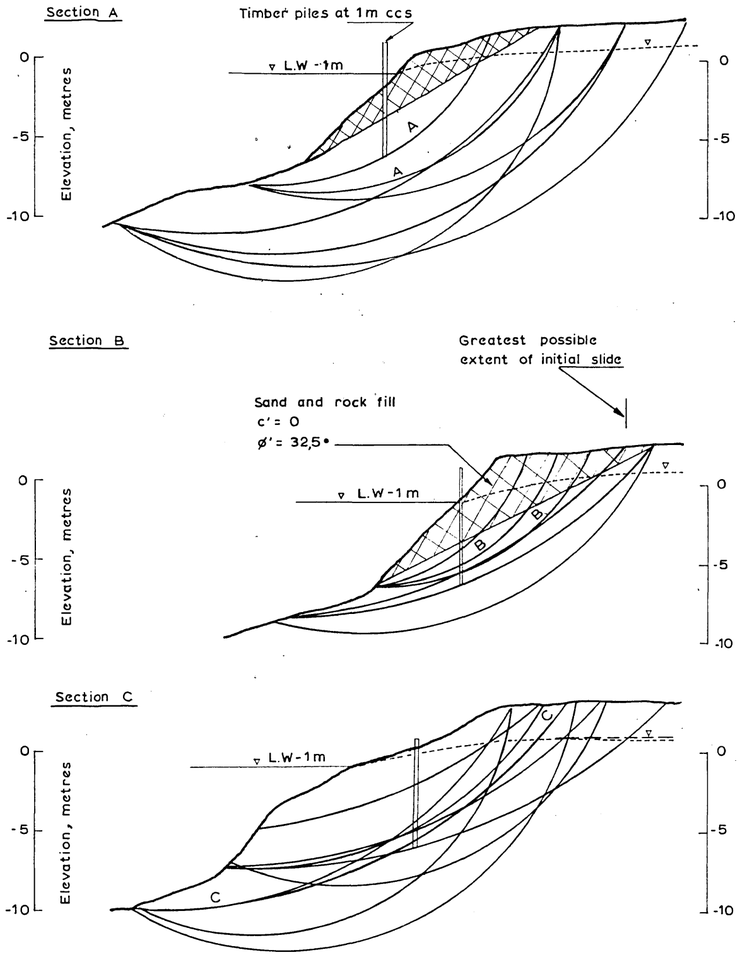
NADDUM SLIDE. SITE CONDITIONS,
SOIL PROPERTIES AND STABILITY CALCULATIONS

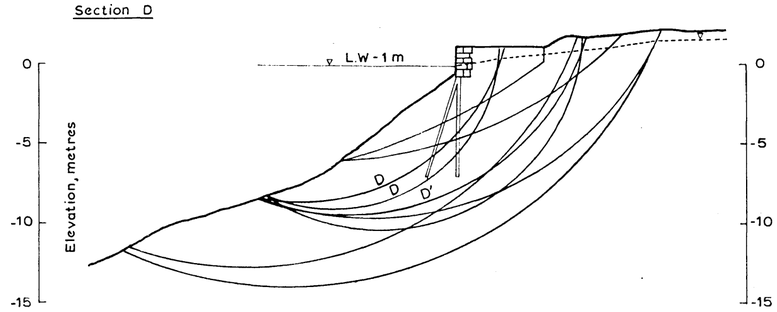
DRAMMEN SLIDE. SITE CONDITIONS, SOIL PROPERTIES AND STABILITY CALCULATIONS

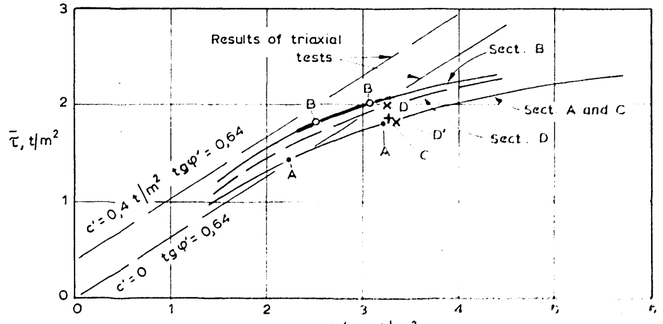


Site plan

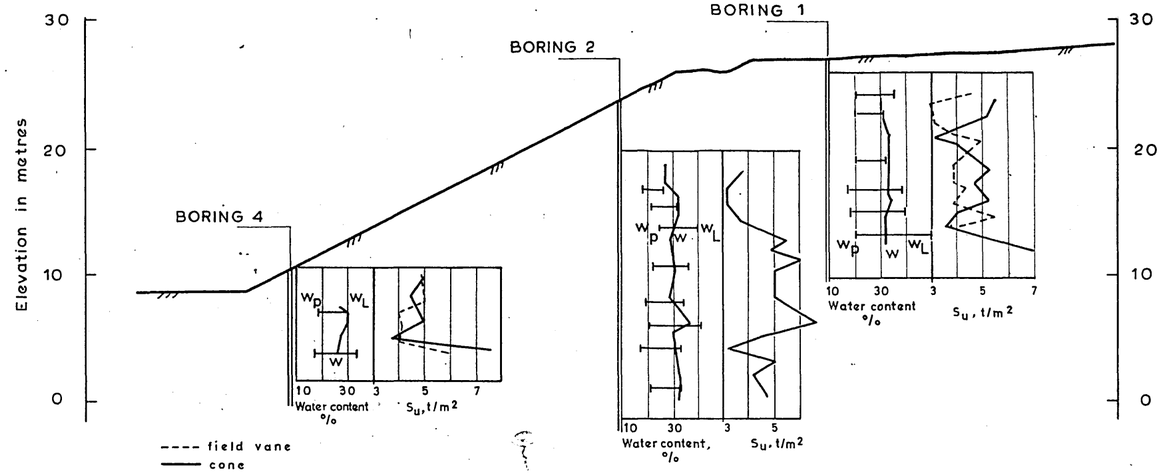




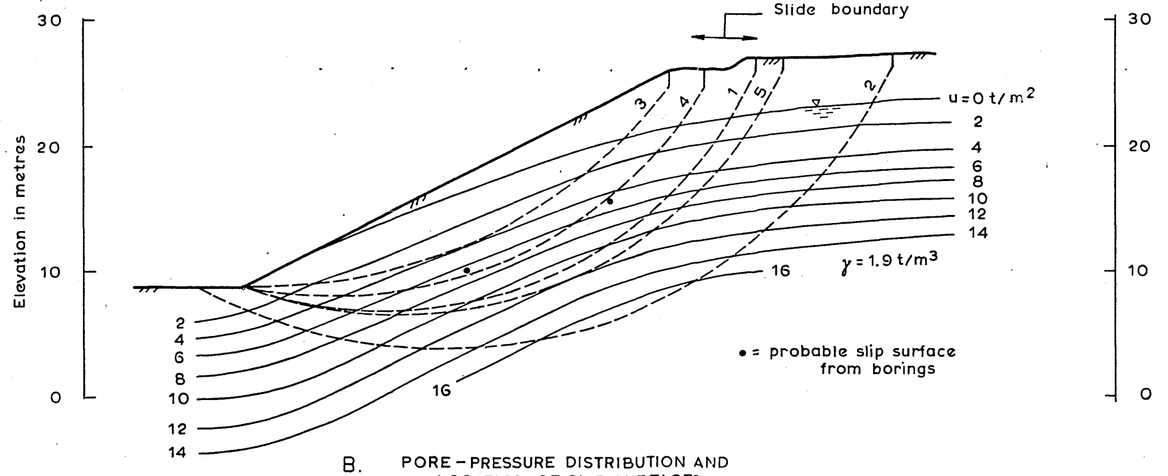




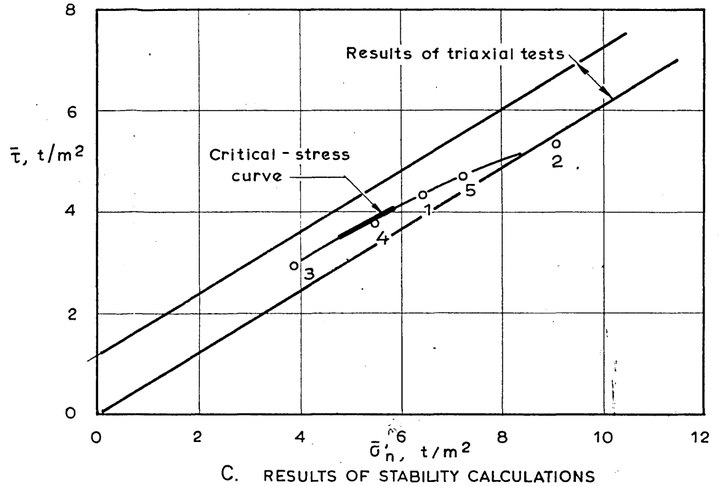
Results of stability calculations



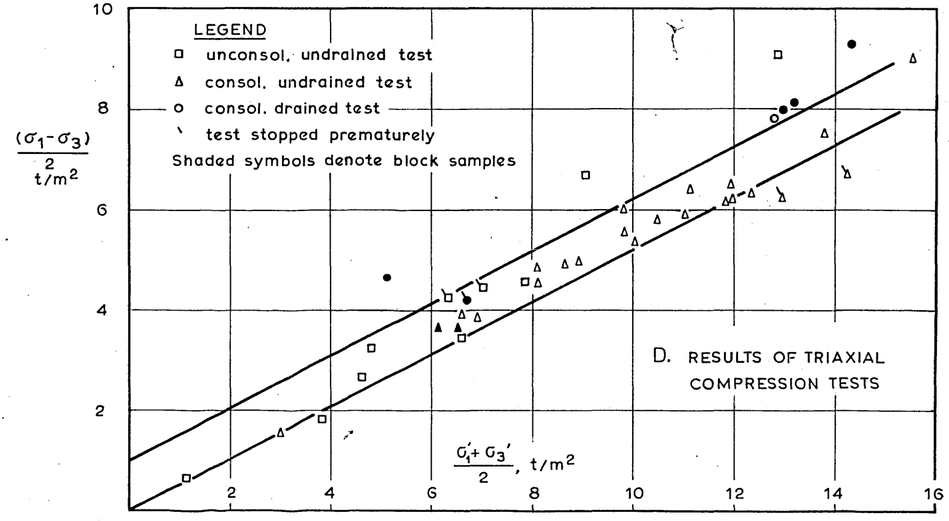
A. SOIL PROPERTIES



B. PORE-PRESSURE DISTRIBUTION AND LOCATION OF SLIP SURFACES

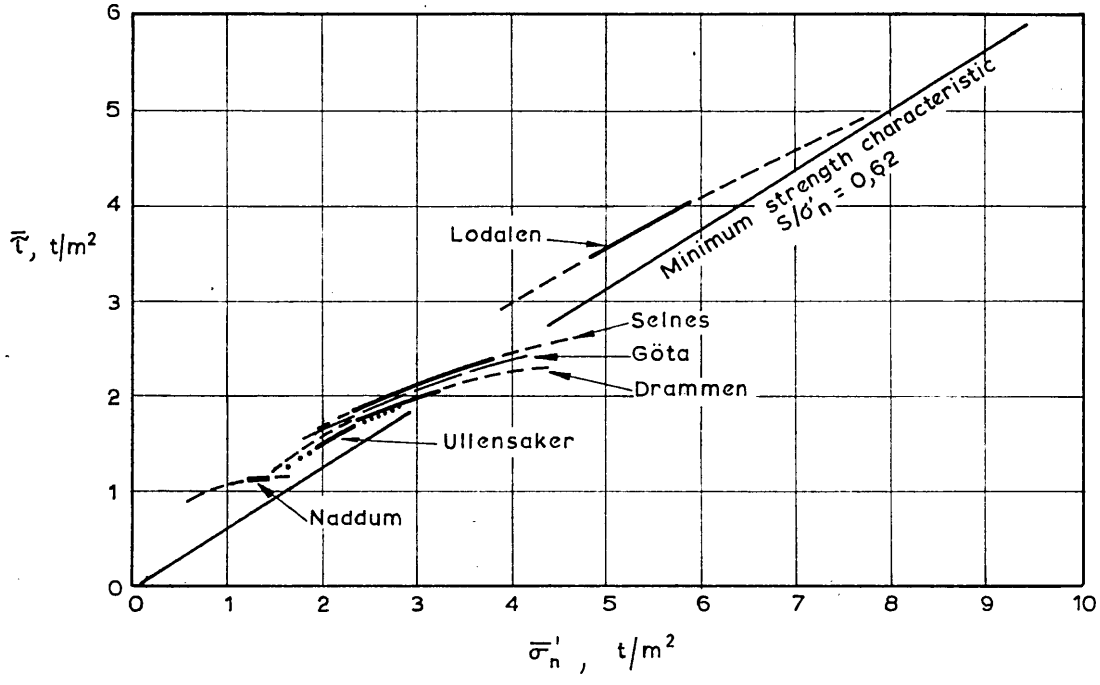


C. RESULTS OF STABILITY CALCULATIONS



D. RESULTS OF TRIAXIAL COMPRESSION TESTS

LODALEN SLIDE. SITE CONDITIONS, SOIL PROPERTIES AND STABILITY CALCULATIONS



SHEAR STRENGTH PARAMETERS OF SOME SCANDINAVIAN POST-GLACIAL CLAYS FROM STABILITY ANALYSES OF INITIAL SLOPE FAILURES

Chapter 4. Physico-Chemical Properties of Minerals and Their Influence on Shear Resistance and Compressibility.

1. Composition of Quick Clay.

Most quick clays are located in sub-polar glaciated regions of the northern hemisphere, and they are composed of fine-grained particles from glacial debris that have been separated, sorted and transported by fresh-water streams, finally to be discharged into marine waters where they flocculated and sedimented. In grain size these clays consist primarily of particles smaller than 60μ (0.060 mm, upper limit of the silt range), which, for comparison, is the approximate upper limit for milled flour. Because glacial debris in Norway is mostly the product from crushing pieces of igneous bedrock, the individual particles composing Norwegian quick clays are pieces of minerals (in both unaltered and weathered states) that composed the bedrock from which the glacial till was derived. The minerals are the massive minerals quartz, feldspar, hornblende and calcite and the layer-lattice minerals mica, hydrous mica and chlorite. The massive minerals quartz and feldspar compose most of the grain-size fraction from 60 to 2μ in quick clays, and below this range the layer-lattice minerals of the mica group predominate. The mineralogical compositions of some quick clays are given in Table 4-1.

Fig. 4-1 contains some photographs of particles typical in quick clays. One photograph shows the minus 60μ fraction of a till that consists of particles of quartz and feldspar, typical for the coarse fraction of quick clays. These particles are angular, either because they are the original crystals in the parent igneous rocks (Smalley, 1966) or because they are the products of crushing of larger crystals. Particles of this size are too small to be abraded during water transport and therefore it is to be expected that all particles of massive minerals in quick clays are angular in form.

Photographs are also presented of the minus 2μ fraction of Selnes quick clay. In these photographs the particles are shadowed by

a beam of light inclined at 1 : 4, so that shadows are cast equal in length to 4 times the height of any obstructions (the short bright shadows in the photographs are features of the apparatus and not of the particles). It is easy to distinguish between the thick particles of massive minerals and the thin flakes of micaceous minerals. The results of X-ray diffraction studies indicate that in this sample, and for most Norwegian quick clays, the micaceous minerals are predominantly mica and hydrous mica, accompanied by some chlorite. Some of the large micaceous flakes are obviously terraced along cleavage planes. The faces of many of the small flakes appear to be complete cleavage planes, but the soft-looking, rather diffuse edges of these particles indicate a gradual thinning of the particles toward the edges, presumably as step-like cleavage terraces. Hence, even these micaceous flakes should be considered as being angular, at least along the edges.

2. System Chemistry and Residual Shear Strength.

Method. It has been mentioned previously that very small values of drained shear-strength parameters have been reported for quick clay and for loose sand. There are several explanations for these results, and the possibility that will be explored in this chapter is that a natural soil, composed of different mineral particles, could exhibit small drained strengths because one of the mineral components has very small strength properties that dominate the behaviour of the soil. Attention will therefore be concentrated on the strength properties of aggregates of pure minerals and mixtures of pure minerals. The main differences between such materials and natural quick clays are that quick clays possess a natural soil fabric and inter-particle cementation bonds. The first factor generally means that the porosity of a natural quick clay is larger than that of an artificial sample, and the second factor means that the strength properties of a natural quick clay are dependent on cementation bonds, cohesion and friction, whereas those of an artificial sample are dependent only on cohesion and friction, these terms being more rigorously defined later. If it were not for the cementation bonds, natural quick clay would behave in drained shear

tests in similar fashion to a very loose aggregate of individual clay-mineral and massive-mineral particles, and the shear resistance would increase steadily until the ultimate or residual value (Skempton, 1964) was reached. If the same soil was tested, beginning from a much smaller porosity, the shear resistance would increase to a maximum value and would then decrease to the same residual value. Hence, for a very loose uncemented soil the drained shear strength, the minimum drained shear strength and the residual drained shear strength are one and the same thing, and it is equal to the residual strength of any sample of the same soil independent of the initial porosity. In the case of cemented soils, if at some stage in a test the cementation bonds are broken more rapidly than the sum of frictional and cohesive resistance is developed, a temporary reduction of strength, or local failure, will result, possibly at a shear resistance less than the residual shear resistance. Obviously, if the test were continued, the shear resistance would ultimately rise to the residual strength value.

The program that has been started had as its purpose the determination of the residual-strength characteristics of the component minerals in quick clays, in different chemical states, and also of mineral mixtures. The following test method was adopted (Kenney, 1966a). All materials that were tested contained primarily particles smaller than 60μ. Samples, 80-mm diameter and about 1 mm thick, were consolidated in the direct-shear apparatus described by Bjerrum and Landva (1966) between two filter plates (carborundum stones) that were prevented from tilting. A ring confined the sample during consolidation. For very fine-grained materials, the filter plates were covered with cellulose filter sheets. The bottom filter plate was kept flooded with water having the same chemical composition as the pore fluid in the sample being tested. The test was performed by shearing the sample back and forth 2 mm from centre until the shear strength became constant at its residual value. The rates of strain that were used varied from 1 mm/hr to 1 mm/7 hrs to allow complete pore-pressure dissipation. It will be mentioned later that within this range of strain rates the magnitude of residual strength for any material

remained essentially constant (see Fig. 5-2). Slipping between sample and filter plates did not occur with any of the materials that were tested.

Massive Minerals. Samples were tested of quartz, feldspar and calcite, the most important massive minerals in Scandinavian soils, that had the characteristics listed in Table 4-1. These were crushed minerals, the largest grains being less than 60μ and containing 1 to 2% by weight less than 2μ. Additional samples of very fine-grained quartz and feldspar were prepared by removing and retaining the minus 2μ fraction of the bulk sample. All samples were mixed and tested in distilled water. The test results are given in Table 4-1 and Fig. 4-2 and lead to the following conclusions:

(a) Residual shear resistance, $\text{tg}\,\varphi_{res}$, of crushed quartz, feldspar and calcite are equal. (The quartz exhibited stick-slip behaviour, and the maximum or static resistance has been plotted in Fig. 4-2).

(b) $\text{tg}\,\varphi_{res}$ is independent of grain size.

(c) Residual shear strength is directly proportional to normal stress.

The crushed materials consisted entirely of sharp angular particles, and to investigate the possible influence of grain shape (Jakobson, 1957) two additional samples were obtained that consisted of more rounded grains; one sample was a dune sand from the Sahara Desert and the edges of the particles were rounded but the general shape of the grains was angular, and the other sample was smoke dust from a ferrosilisium smelter and the particles were absolutely spherical and consisted of amorphous SiO_2 (chemically equivalent to quartz). Photographs of the two materials are given in Fig. 4-1. The test results (Table 4-1 and Fig. 4-2) indicate smaller values of $\text{tg}\,\varphi_{res}$ than for the strongly angular crushed materials. The Sahara sand can be considered as providing the lower limit of angularity and of residual shear strength for the massive minerals contained in Norwegian clays. The SiO_2 probably provides the absolute lower limit of both angularity and grain

size for massive minerals, and its large residual shear strength of tg φ_{res} $\approx$ 0.56 probably is also the lower limit of strength for an aggregate of massive minerals composed of quartz, feldspar or calcite.

The range of tg φ_{res} for crushed massive minerals and Sahara sand is from 0.69 to 0.58 at a normal stress of 1 kg/cm^2, corresponding to φ_{res} values of 35 to 30 degrees, and these values were obtained after considerable strain in drained tests. They are in agreement with most published data on granular materials, with the exception of the results of drained triaxial tests on loose sand presented by Bjerrum, Kringstad and Kummeneje (1961) and the result of a drained direct simple-shear test on loose sand presented by Bjerrum (1961), that gave minimum values of tg φ_d = 0.36 and 0.29, respectively (φ_d = 20 and 16 degrees). These tests were of considerable importance to this thesis because on the basis of the results Bjerrum (1961) drew very important conclusions concerning the behaviours of quick clays and other loose-structured soils.

A careful review was made of the original triaxial test data (Kenney, 1966b) and it was found that all the tests that gave values of $\varphi_d < 26$ degrees were not acceptable. The drained triaxial tests that gave these small values were performed by deforming the samples at constant rate of strain and keeping the axial stress σ_a constant by reducing the radial stress σ_r in steps, and the tests were stopped when $\sigma_a - \sigma_r$ became maximum. When the tests were stopped at $(\sigma_a - \sigma_r)_{max}$, the ratio σ_a/σ_r was still increasing and therefore tg φ_d was still increasing. Hence, all the reported small values of φ_d are underestimates. The smallest value obtained from the acceptable triaxial tests is tg φ_d = 0.5 ($\varphi_d \approx 26$ degrees). The small value of φ_d obtained from the direct simple-shear test was more remarkable than the triaxial-test results because the shear resistance of the sample actually decreased from a maximum of tg φ_d = 0.29 to a value of tg φ_d = 0.23 (φ_d = 13 degrees) when the test was stopped. From the original test records much information can be found that indicates that this test could be inaccurate; for example, this test was one of the initial series performed with a prototype apparatus that still had many short-

-comings and also a remark was found in the records that a water lead became bent at a certain stage of the tests and prevented water from leaving the sample, this providing the possibility of the existence of excess water pressures during some of the tests. Although these are possibilities, no definite evidence could be found on which this test could be judged unacceptable. However, in several later test programs on similar, but not identical, sand, using improved apparatus and testing techniques, and obtaining very large initial porosities, the minimum shear resistance that was ever obtained was $\text{tg}\,\varphi_d = 0.51$ ($\varphi_d = 27$ degrees), and this is a strong indication that the very small value of φ_d reported by Bjerrum is not reliable (Kenney and Sörlie, 1966).

Layer-lattice Minerals. The next group of minerals that was tested was the layer-lattice group, frequently referred to as clay minerals when the particles are small in grain size. Those that are of greatest importance to this study are mica and hydrous mica because they are the predominant materials in the minus 2μ range of particle sizes in Norwegian clays and compose about 50% of the total dry weight. The mica found in these clays is primarily muscovite, and it can degenerate by loosing potassium to become hydrous mica, then illite, until complete degeneration would result in montmorillonite. Whereas muscovite and montmorillonite are distinct minerals, hydrous mica and illite (and others) are only intermediate phases of degeneration or generation between these two mineral types. Another constituent of most Norwegian clays is the clay mineral chlorite, but it has not been tested in this study.

One of the most important common characteristics of the clay minerals mica, hydrous mica and montmorillonite is their surface reactivity. Due to atomic-lattice imperfections within the particles, the cleavage surfaces have a negative charge. The edges of the particles are not cleavage surfaces but are fractures across the lattice layers, and due to broken valence bonds the edges have a net positive charge that, for the clay minerals in question, is considerably smaller than the negative surface charge. In the presence of an electrolyte, the positively-charged ions, cations, are attracted to the negatively-charged cleavage surfaces, and the negatively-charged ions, anions, are attracted to the

positively-charged edges. The cations are loosely held by long-range forces originating from within the particle, and they can be exchanged by other ions depending on relative concentration, valence, polarization and size of the different ions.

The reason why system chemistry must be taken into account in the study of quick clays is that they have experienced in their histories important changes of system chemistry. They are marine deposits, originally sedimented in strong solutions of NaCl. In this environment the negative potential at the faces of the clay-mineral particles was satisfied by Na cations and the particles became converted to the Na-state. Subsequently, when the soil was leached by fresh water, the NaCl concentration in the pore fluid was reduced, but the clay particles remained in the Na-state. Because the ion concentration in the pore fluid was then small, the addition of small quantities of other cations to the system could cause the replacement of sodium cations on the particles, and one such exchange that is common in nature is the replacement of sodium by potassium being released during the weathering of mica and feldspar. To simulate in the laboratory these changes that occur in natural soils, the clay minerals that were to be shear tested were prepared in several different homoionic states (state in which there is only one type of cation present), such as sodium, potassium and calcium, with different ion concentrations in the pore fluid. In addition, some samples were prepared from the sodium state by the addition of the dispersant Calgon. The action of Calgon will be considered later. The method of preparation of the clay minerals is described in Appendix 1.

Most of the tests were performed on three clay minerals; muscovite, hydrous mica[x] and montmorillonite. The muscovite was purchased in a crushed state. The hydrous mica was the fine fraction of

x) The term hydrous mica will be used where, in reality, it would be more correct to use mica/hydrous mica. In X-ray diffraction tests the material is identified by a broad-based 10 Å reflection, but the base of the reflection is insufficiently broad to indicate illitic minerals.

the natural Selnes quick clay and contained 10% massive minerals and 5% chlorite. The montmorillonite was the fine fraction of a bentonite sold under the trade name KWK. The characteristics of the various preparations of these clay minerals and the results of the shear tests are given in Table 4-1 and Fig. 4-2. The following conclusions can be drawn from the results of the tests:

(a) The minimum shear resistance of hydrous mica was obtained for the sodium, low salt-content state (the state in which quick clays exist). At a normal stress of 1 kg/cm^2, $\text{tg}\,\varphi_{res} \approx 0.3$, corresponding to $\varphi_{res} \approx 17$ degrees. For the sodium high salt-content state (marine clay before leaching) the residual strength corresponded to $\text{tg}\,\varphi_{res} \approx 0.45$ or $\varphi_{res} \approx 24$ degrees, and for the potassium low salt-content state (weathered leached clay), $\text{tg}\,\varphi_{res} \approx 0.39$ or $\varphi_{res} \approx 21$ degrees.

(b) The behaviour of the clay minerals was found to be strongly influenced by system-chemistry changes. Increases of residual shear strength were obtained for (i) increases of electrolyte concentration, (ii) use of cations having larger valencies ($Ca^{++} > K^+ = N_a^+$), (iii) use of cations having greater polarization (K > Na) and smaller size in the hydrated state ($K < N_a$).

(c) The factors listed under conclusion (b) also caused increases of plasticity and residual water content of muscovite and hydrous mica. The opposite results were obtained for montmorillonite.

(d) Calgon caused a decrease of the residual strength of all the clay minerals, and changed the plasticities and residual water contents in the same direction as that caused by a decrease of electrolyte concentration.

To place the last three conclusions into a mechanistic framework, it will first be necessary to consider the physico-chemical properties of clay minerals. This subject is large and complex, and a detailed treatment of it is beyond the scope of this thesis. A brief and oversimplified picture will be given, and for more details the following references are recommended; Rosenqvist, 1955; Lambe, 1960; Grim, 1962; van Olphen, 1963. The clay-mineral particles have excess negative charges existing within their faces and a number of unsatisfied positive charges existing around their edges. Cations are attracted to the negatively-charged surfaces, but these ions also desire to move away from each other due to electrostatic forces of repulsion and thermal energy, and therefore the actual positions they occupy are compromises between these forces. The thickness of the resulting diffuse ionic layer around the clay particles is the distance required to neutralize the net charge on the particle; that is, the distance over which there is an electric potential around the clay particle. When the cation concentration in the pore fluid is increased there are more cations available to satisfy the surface-charge of the particles, and therefore the thicknesses of the diffuse layers of ions and water around the particles decrease. Decreases of thickness also would be obtained if the cation concentration remained the same and if cations were used that had larger valencies, higher degrees of polarization, or smaller hydrated size. All these factors increase the ability of the cation to satisfy the surface charge and/or to move more closely to the particle and to other cations. In principle, the same explanation applies to the edges of the particles, except that the charges to be satisfied are positive and that this is accomplished by anions.

There are essentially four types of forces that influence the normal stress transferred between clay-mineral particles. The first is the externally-applied effective-stress force caused by the weight of the soil or by an applied load. The second force is the van der Waals force of attraction that is important when the distances between particles are small, but that decreases very rapidly for increasing distances. The third force is one of electrostatic attraction between unlike charges on the faces and edges of different particles. This force is sensitive to changes of inter-particle distances and is influenced by changes of system

chemistry, possibly increasing for increasing electrolyte concentrations and increasing for increasing polarizability of the cations. The fourth force is the electrostatic force of repulsion between similar diffuse ionic layers. This force is sensitive to changes of inter-particle distances and is greatly influenced by changes of thickness of the diffuse ionic layers as the result of changes of system chemistry mentioned previously.

If these forces act over a unit of area, the net result of all the stresses can be defined as the absolute inter-particle normal stress, σ'_{an}:

$$\sigma'_{an} = \sigma'_{n} + (A - R) \quad . \quad . \quad .

In natural soils, as opposed to remoulded soils, there can exist interparticle bonds caused by the cementation action of chemical precipitates, and possibly bonds caused by the welding together of particles under stress (recrystallization). These bonds can be imagined as being brittle and failing at small strains. Such cementation and weld bonds will be denoted by the symbol b_c.

Let it now be assumed that the shear resistance per unit area, s_a, developed in a particulate mass is equal to the sum of the cementation-bond strength b_c, linkage-bond strength, b_1, and the product of the absolute normal stress, σ'_{an}, and the average absolute coefficient of friction[x] between soil particles, ψ_a, that will be assumed to be independent of system chemistry.

For natural soils : $$s_a = b_c + b_1 + \sigma_a' \cdot \psi_a$$
$$= b_c + b_1 + [\sigma' + (A-R)]\psi_a \quad . \quad . \quad . \quad . \quad 4.2$$

For uncemented soils : $$s_a = b_1 + [\sigma' + (A-R)]\psi_a \quad . \quad . \quad . \quad . \quad 4.3$$

In these equations, the terms b_1 and (A-R) are similarly affected by most system changes. For example, the use of polyvalent cations will increase b_1 and decrease R, and result in increases of both b_1 and (A-R). Similar changes would occur for the presence of cations of increasing polarizability. Reductions of ion concentration would increase R and hence decrease (A-R) and if it had a direct influence on b_1 this effect could be imagined as decreasing b_1. Most of these factors cannot be quantitatively

x) The term friction is used to describe that shear resistance between two particles that is proportional to the normal force acting between the particles, independent of the fundamental cause of the resistance.

described, and equation 4.3 does not provide, for example, an explanation of why all minerals and soils exhibit a residual shear strength that is nearly directly proportional with applied stress, but it will be shown that it provides a qualitative explanation of the effects of system-chemistry changes.

Referring back to the conclusions drawn from the test results on uncemented soils, it has been explained that the three factors listed in conclusion (b) reduce the repulsion stress R and can increase the attraction stress A and the linkage-bond strength b_1, the net result being that the terms (A-R) and possibly b_1 increase in magnitude and hence from equation 4.3 the shear resistance increases, as measured in the tests.

Conclusion (c) concerned increases of plasticity and increases of residual water content that were dependent on the three factors listed in equation (b). These three factors cause an increase of the term (A-R) and possibly b_1 and enhance edge-to-face association by electrostatic and van der Waals forces and also face-to-face association by van der Waals forces. In the case of muscovite and hydrous mica the edge-to-face association or cardhouse structure is the most desired by the particles, and therefore an increase of the term (A-R) and b_1 causes an increase of shear resistance between particles. The liquid limit is the water content at which a soil exhibits a specific dynamic shear strength (Kenney, 1963), and if the soil exists at its liquid limit and system changes are made such that (A-R) and b_1 are increased, its strength will increase and water will have to be added to reduce the strength again to the specific value for the liquid-limit test. In the case of montmorillonite, in which the particles are very active and have very thick diffuse ionic layers in comparison to their own thicknesses, an increase of the term (A-R) causes the particles to move together to the extent that at large electrolyte concentrations or in the presence of polyvalent cations the montmorillonite particles have grouped themselves tightly into pockets or aggregates of parallel face-to-face particles. To do this there must be water expelled from between the aggregating particles. Although the reduction of the

repulsion forces and an increase of b_1 increases the shear resistance that can be developed between particles, the amount of water expelled from within the particle aggregates has the greatest effect. Therefore, increasing the term (A-R) by any of the three methods listed in conclusion (b) will have the same effect as adding water, and in the case of montmorillonite the water comes from within the soil itself rather than from outside.

The residual water content in a shear test depends on the shear resistance that can be developed at the particle contact points to resist volume change, and it also depends on what fabric the soil particles desire for a particular system chemistry. These same factors governed the water content at the liquid-limit strength, and therefore residual-strength water content and liquid-limit water content will vary similarly for system changes.

Conclusion (d) concerned the effects of Calgon. Calgon is a trade name for sodium hexametaphosphate. In solution it dissociates into sodium cations and long chains of negatively-charged phosphate anions. The sodium cations are attracted to the faces of the particles. The large-valence hexametaphosphate anions are attracted to the positively-charged edges of the particles and they react with exposed aluminum in the layer-lattice structure to become chemisorbed in the form of complex anions. Each large-valence anion is attached to a particle at only a fraction of its ionic groups, and the remaining excess negative charge gives the edge of the clay-mineral particle a net negative charge. In this way the whole particle surface becomes negatively charged, eliminating any electrostatic forces of attraction that existed prior to Calgon treatment. Therefore, in terms of the shear resistance equation 4-3, the stress A has been reduced considerably, and R and b_1 have remained relatively constant (depending on changes of the Na^+ concentration in the pore fluid), and the net result is a decrease of (A-R) and a decrease of s_a.

Mineral Mixtures and Natural Soils. Although the residual-strength shear tests showed clearly that the minimum shear resistances of the mineral components of quick clay were large, the results were not

proof that mineral mixtures could not exhibit strengths that were smaller than those of the component minerals. A series of tests were made on quartz-clay mineral mixtures, and the results are given in Fig. 4-3. Quartz was chosen to represent the content of massive minerals. The results clearly show that the tg φ_{res} of the mixture varies smoothly between the values of tg φ_{res} of the two minerals in the mixture. For all the mixtures tested it was found that tg φ_{res} of the mixture never became smaller than tg φ_{res} of the clay mineral in the sodium, zero-salt state.

Also plotted in Fig. 4-3 are the results of tests performed on remoulded natural clays, one group being quick clays from Norway and Canada that are composed primarily of massive minerals and the clay minerals hydrous mica and chlorite, and the other group being clay shales that have reasonably large contents of the low-strength mineral montmorillonite and mixed-layer minerals containing montmorillonite. The quantitative mineralogical compositions of the soils were determined by chemical, DTA and X-ray diffraction methods (Appendix 2) and the results are summarized in Table 4-1. To plot the results of the tests on quick clays in Fig. 4-3 the sum of the contents of the clay minerals chlorite, hydrous mica and mixed-layer montmorillonite was taken as the content of mineral A. In the case of the clay shales, the montmorillonite group of minerals would have greatest influence on the behaviour of the soil, and therefore the sum of the contents of montmorillonite and mixed-layer with montmorillonite was used to plot the test results.

The two soil groups plot separately in Fig. 4-3 and are in reasonable agreement with the test results for artificial mixtures. The quick clays, without exception, plot above the mixture curves, and the reason for this is not known yet. It could be due to inaccuracies in the determination of the mineral composition of the soils. It could be due to the soils being in chemical states quite different from the sodium state, the possibility being that weathering in the laboratory had occurred; such an effect should have altered the plasticity of the clays, but this does not seem to have happened. Another possibility is that grain-size distribution is of importance in those natural clays that have a progressive change in

grain size, such as most micaceous clays. Thus, in the artificial mixtures of hydrous mica, the quartz might have been too coarse and too narrowly graded. For soils containing montmorillonite, the montmorillonite is usually extremely fine grained, and it is possible that such natural soils have discontinuous grading curves somewhat similar to those of the artificial mixtures that were tested.

Although the agreement between tg φ_{res} of natural quick clays and tg φ_{res} of mineral mixtures is not particularly good, and although the reason for this is not yet known, the results of these tests indicate that the minimum drained strength of uncemented quick clays, and their mineral components, is not less than tg $\varphi_{res} = 0.30$.

Conclusions.

(a) It has been explained that the minimum drained shear strength of an uncemented soil, independent of porosity, is equal to the residual shear strength at large strains. Therefore, to determine the minimum drained strengths of uncemented quick clays and loose sands, and their mineral components, a program of residual-strength tests was performed.

Tests on massive minerals indicated a range of tg $\varphi_{res} \approx 0.58$ to 0.69, tg φ_{res} depending primarily on the angularity of the particles, possibly on the grain-size distribution, but not depending on absolute grain size. These results were in conflict with small tg φ_d-values reported by Bjerrum (1961) and Bjerrum et al (1961) and checks were made of the original test data. By eliminating those tests that were in error, it was found that, for the sands tested, tg $\varphi_d \geq 0.5$, which is in general agreement with the residual-strength tests. Also of importance was the observation that in all the acceptable NGI triaxial and direct-shear tests on sand there did not occur a premature or local-type failure at a stress level less than tg $\varphi_d = 0.5$.

(b) Hydrous mica is the predominate clay mineral in quick clays and in residual-strength tests its minimum strength corresponds to $tg\,\varphi_d = 0.30$. This result is little affected by grain size (it increases for decreasing grain size). The residual strength of clay minerals is very dependent on system chemistry and strength increases result for increases of ion concentration, increases of the degree of polarization of the cation, increases of cation valence, and decreasing size of the hydrated cation.

(c) Tests on mineral mixtures support the results of tests on pure minerals, indicating that the minimum drained strength of quick clay is $tg\,\varphi_d > 0.3$. Tests on remoulded quick clays gave $tg\,\varphi_{res} \geq 0.5$, a value considerably larger than that expected from the results of tests on mineral mixtures, and the reason for this finding cannot be given.

4. System Chemistry and Compressibility.

If changes of system chemistry have influences on the shear-resistance properties of clay minerals, it might be expected that they could also have influences on the compressibility of soils because compressibility is governed by the shear resistance that can be developed between particles. If the shear resistance between particles could be increased the soil would become less compressible, and if the shear resistance between particles could be decreased the soil would become more compressible.

Of primary interest in the study of quick clays was the behaviour of a marine clay during the reduction of salt content in the pore fluid. From the experiments concerning residual shear resistance it was found that a decrease of the electrolyte content would cause an increase in the thickness of the diffuse ionic layers, an increase of the repulsion forces between clay-mineral particles, and a reduction of the shear resistance that could be developed between two clay-mineral particles. Therefore, it could be expected that leaching would cause decreases of shear resistance at particle contacts, and at those particle contacts where the shear resistance was reduced below the level of the inter-particle shear stress acting before

leaching shear strain or slipping would occur and this would be manifested as volume compression. On the other hand, if at the particle contacts there existed a reserve of shear strength that was greater than any strength reductions that could occur due to system changes by leaching, the process of leaching would not cause compression but it would lower the available strength reserve at the particle contacts, and for increases of applied load the sample would exhibit smaller values of "apparent" preconsolidation pressure P_c' and larger compressibilities. Conversely, if the electrolyte concentration of a sodium clay was small, such as in a natural quick clay, and the electrolyte concentration was increased, then this should result in a stronger, less-compressible clay than the original one. Four test programs were made to investigate the influence of system-chemistry changes on compressibility. Three of these programs concerned the effects of reduction of ion concentration, and they will be described in this section. The fourth program concerned the effects of system changes on a cemented quick clay, and it will be described in the following section.

The first program was performed on undisturbed samples of Manglerud III quick clay. Samples were built into 50 cm^2 and 2 cm high oedometer cells having top and bottom pieces similar to those in Fig. 2-3 that enabled the sample to be permeated by fluid by using a 1-metre length of suction tube hanging from the oedometer cell to the floor (Fig. 4-4). The samples were loaded to approximately the natural overburden pressure, and at this stage they were permeated with a solution of 30 gm NaCl/l to raise the salt content to about the level it was during natural sedimentation. The samples were then loaded to pressures corresponding to $P'/P_o' \approx 1.3$, 1.7 and 2.3, and after a period of 4 days they were leached with a solution of 0.5 gm NaCl/l. As shown in Figs. 4-5 and 4-6 large compressions resulted from the decrease of the electrolyte concentration in the pore fluid. Of special interest is the behaviour of test Ø 6, Fig. 4-6. Due to air in the seepage leads, there was a period during the leaching process when little fluid passed through the sample. At this time the salt content of the effluent was about 10 gm NaCl/l and therefore the average salt content of the sample was somewhat higher

than this value (seepage occurs through the most open channels in the soil). Rather little compression occurred during this period. Very shortly after seepage was resumed, and when the salt content decreased below about 10 gm/l compression occurred. This same general behaviour was found in all tests and it agrees well with Rosenqvist's finding (Rosenqvist, 1955) that changes of NaCl concentration above 10 gm/l have little effect, whereas changes below 10 gm/l can have large effects on the behaviour of unweathered Norwegian clays.

The results of these tests showed clearly that decreasing the electrolyte concentration from 30 to about 1 gm NaCl/l caused considerable compression of the samples at axial stresses greater than P_o'. As shown in Table 4-2, leaching also caused significant decreases of plasticity of the soil from its high-salt state ($w_L \approx 45\%$, $w_p \approx 22\%$). All three samples were quick upon completion of the tests, in spite of the fact that they all had experienced large compressions.

This test series left unanswered several questions: (i) what would be the behaviour of a natural salt clay subjected to leaching, (ii) would samples, subjected to axial stresses less than the overburden pressure, compress due to leaching, or perhaps even swell, and (iii) what would be the effect of increasing the salt concentration again after the sample was first leached? To answer such questions a second series of tests were performed on the Sköyen clay, a natural normally-consolidated salt clay, that has been briefly described in Chapter 2. The results of these tests are given in Table 4-2 and Figs. 4-5, 4-7 and 4-8. Samples were loaded to axial stresses both smaller and larger than the existing overburden pressure P_o', and after periods of constant axial stress, during which the secondary-compression curve could be determined (usually about 2 weeks), the samples were leached and later loading was continued.

The results are generally similar to the first series of tests on Manglerud III clay, but the compression due to leaching was very much smaller for the Sköyen samples than for the Manglerud III samples. If the water contents of these samples before leaching are considered with

respect to their plasticities after leaching, it is found that for Manglerud III, $I_L > 4$, and for Sköyen, $I_L \approx 1$, with the exception of sample Ø 122 for which $I_L \approx 1.9$ and which experienced the largest compression of the Sköyen samples. It appears that the susceptibility of a clay to large compressions upon leaching is dependent on the degree to which leaching can decrease the plasticity characteristics of the soil in relation to the water content of the soil. If the resulting liquidity index is less than unity, little compression will occur. If the resulting liquidity index is much larger than unity and the soil structure is not held together by strong cementation bonds, large compressions will occur; if the soil is strongly cemented, small compressions will occur. Butextending this line of thought, it can be postulated that those clays in nature that possess both very large liquidity indices and sensitivities are cemented clays, but proof of this is as yet very limited.

The results of the tests indicated that the apparent preconsolidation pressure P_c' was equal to about $0.8 P_o'$ (indicating sample disturbance), and that at a stress level less than P_c' (test Ø 121), leaching caused compression (Fig. 4-7). At stress levels above P_c' the leached samples did not exhibit any quasi-preconsolidation upon reloading (Leonards and Ramiah, 1960), except the special test Ø 126 that, after leaching, was permeated with a solution of 30 gm NaCl/l before loading was resumed. It will be observed in Fig. 4-8 that the rate of secondary compression of sample Ø 126 diminished as soon as the salt concentration in the pore fluid increased.

The third series of tests have been described in Chapter 2 and the results are given in Table 2-2 and Figs. 2-5 and 2-6. These were tests on the Sköyen clay in which leaching was performed at axial stresses below and above P_o'. The compressibility behaviours of the samples were similar to those described above and require no further comment. In addition to measuring volume changes, the radial stresses in the samples were also measured and it was found that in all cases small increases of σ_r' resulted from decreases of the electrolyte concentration of the pore fluid. Similar tests have not been performed on clays that exhibit large compressions as the result of leaching; it is possible that for such clays the increase of K_o during leaching would be larger than those shown in

Fig. 2-6. If no compression results from leaching, then there will be no increase of K_o (no strain change, no stress change), independent of whether or not the leached soil becomes quick.

The main results of the three test series can be summarized as follows. Reduction of electrolyte concentration in sodium clays from 30 to about 1 gm NaCl/l caused (1) decreases of plasticity, (2) increases of liquidity index, and hence (3) increases of sensitivity, in spite of (4) volume compression. It was found that (5) the rate of volume compression was accelerated and the compressibility of the soil increased by reduction of salt concentration, and (6) the opposite was observed for an increase of salt concentration from about 1 to 30 gm/l. (Better evidence of this is presented in the following section). Finally, (7) reduction of salt concentration at constant axial stress caused an increase of σ_r'.

Points 1 to 3 require no comment, having previously been discussed. The remaining points will be explained with the use of Fig. 4-9. Two methods of explanation will be used, Method 1 being based on the experimental fact that reduction of electrolyte concentration leads to a decrease of the shear-resistance potential of clay minerals, and Method 2 being based on shear resistance in terms of average absolute stresses at the particle contacts.

Soil compression in the oedometer is primarily due to slippage or shear failure at the contacts between particles; this sub-microscopic shear failure is not to be confused with macroscopic shear failure in other types of soil test. In diagram A is given a stress plot and a stress-compression plot, and in the stress plot two K_o curves for normally-consolidated clays are drawn, one for high salt and one for zero salt. The high-salt curve represents sub-microscopic failure at higher shear stresses than the zero-salt curve because high-salt clay minerals have larger shear- strength properties than zero-salt clay minerals. In diagram A it is supposed that a high-salt sample exists at stresses less than failure, and upon increasing the axial stress σ_a' the soil follows the indicated stress and stress-deformation paths. If, however, the salt content had been reduced before loading began, upon loading the weakened clay would not be able to withstand as high shear stresses as the high-salt clay, with the

result that it would follow a lower stress path and a larger-compression curve. In diagram B the high-salt clay is in a normally-consolidated state and in the stress plot it is located on the failure curve. If the clay is leached the shear resistances at particle contacts are reduced to values less than the acting shear stresses and micro-shear occurs causing compression at constant axial stress σ_a'. Because the shear resistance at the particle contacts decreased at constant σ_a', then the radial stress σ_a' increased and K_o increased. Should salt now be added to the clay its behaviour would be that shown in diagram A for a soil at $\sigma_a'/\sigma_r' <$ failure. If the salt clay is loaded and unloaded until it reaches the failure state, and then leached, the shear resistance at particle contacts will decrease and micro-shear failures will occur, as in the case of normally-consolidated clay. However, rather than compress as it did when leached in the normally-consolidated state, it should swell and the radial stress σ_r' should decrease.

The second method used to explain the test results has as its basic assumption that the clay has a unique K_o-failure envelope defined in terms of absolute stress, where the absolute axial and radial stresses are given by the following equations.

$$\sigma_{aa}' = \sigma_a' + (A-R)$$

$$\sigma_{ar}' = \sigma_r' + (A-R)$$

For increases of salt concentration the thicknesses of the diffuse ionic layers decrease and the forces of repulsion R decrease. Conversely, for decreases of salt concentration, the forces of repulsion R increase. If the fabric of the clay has no preferred orientation (and this is a simplifying assumption), the repulsion forces between clay-mineral particles will statistically act equally in all directions and therefore can be considered as being analogous to a positive pore-water pressure. If the ion concentration is increased, R will decrease, and the absolute effective stresses transferred through the mineral particles will increase isotropically by $\Delta\sigma = -\Delta R$ (more correctly, $\Delta\sigma = \Delta[A-R]$. For decreases of

the ion concentration, the absolute effective stresses transferred through particles will be reduced by $\Delta\sigma = \Delta R$.

Consider now an element subjected to the external effective stresses σ_a' and σ_r'. A measure of the shear resistance mobilized at the particle contacts is the ratio of the average absolute stresses:

$$\frac{\sigma_{aa}'}{\sigma_{ar}'} = \frac{\sigma_a' + A - R}{\sigma_r' + A - R}$$

By decreasing the ion concentration, R is increased and this leads to a decrease of both σ_{aa}' and σ_{ar}' and to an increase of the ratio $\sigma_{aa}'/\sigma_{ar}'$, these changes occurring for no change of the average maximum shear stress $\frac{1}{2}(\sigma_a' - \sigma_r') = \frac{1}{2}(\sigma_{aa}' - \sigma_{ar}')$ (see diagram C). In the leaching tests, both σ_{aa}' and σ_{ar}' decreased, but rather than swelling for decreasing stresses, the samples compressed. This is interpreted as meaning that the decreases of σ_{aa}' and σ_{ar}' were small in their effect compared to the increase of the stress ratio $\sigma_{aa}'/\sigma_{ar}'$, this increase being a measure of the decrease of stability at particle contacts and a decrease of the reserve strength at the particle contacts. Upon load application the ratio $\sigma_{aa}'/\sigma_{ar}'$ would increase until the failure envelope was reached, and it is seen in diagram C that the increase of σ_{aa}' required to reach P_c is less for the zero-salt clay than for the high-salt clay. The condition where changes of salt content are made when $\sigma_{aa}'/\sigma_{ar}'$ = failure is shown in diagram C. In the above discussion mention has not been made of the terms b_c and b_1 in equation 4.2. If these terms change in magnitude they would usually change in the same direction as the term (A-R) and would therefore follow the mechanistic picture that has been presented.

The existence of diffuse layers of ions and water that are fixed to clay minerals has been shown to have effects on strength and plasticity. These layers also have an effect on permeability. In Figs. 2-5, 4-6, 4-7 and 4-8 it is shown that permeability decreases when the electrolyte concentration in the seepage fluid is decreased and this can be explained by an increase of the thickness of the diffuse ionic layers for decreases of ion concentration and hence a decrease of the effective pore size. For increases of electrolyte concentration the permeability should increase,

and this is demonstrated in test Ø 126, Fig. 4-8, in which a salt clay was first leached and later its ion concentration was increased. The changes of permeability clearly show that the diffuse ionic layers directly influence permeability.

5. Compressibility of a Cemented Clay.

In a series of triaxial tests performed on samples of a Canadian quick clay from Labrador, it was found that this clay behaved very differently from Norwegian quick clays having about the same mineralogy, plasticity and water contents (see Chapter 5). It was tentatively concluded that the main reason for the differences of behaviour was that the Labrador clay had a cemented structure, whereas the Norwegian clays were very weakly cemented, if cemented at all. The fourth series of compressibility tests was performed on this Labrador quick clay to investigate the influences of (i) removing possible cementing agents, (ii) permeating the soil with the dispersing agent Calgon, and (iii) increasing the electrolyte concentration in the pore fluid.

Information concerning classification data and mineralogy are given in Tables 4-1 and 4-3. The sample was obtained from a shallow test pit in the back scarp of an old slide and is not representative of the soils at greater depths in the profile because of chemical alterations that are only found in the surface soils. The chemical composition of the pore water, given below, indicates that the clay is in a combined sodium-calcium state. It is known that the clay was originally a marine deposit, and the small salt content of pore fluid is evidence that it has been leached.

Ca^{++}	2.2 m. eqv./l	Cl^-	1.6 m. eqv./l
Na^+	4.4 m. eqv./l	HCO_3^-	5.7 m. eqv./l
K^+	0.7 m. eqv./l		
Total	7.3 m. eqv./l	Total	7.3 m. eqv./l

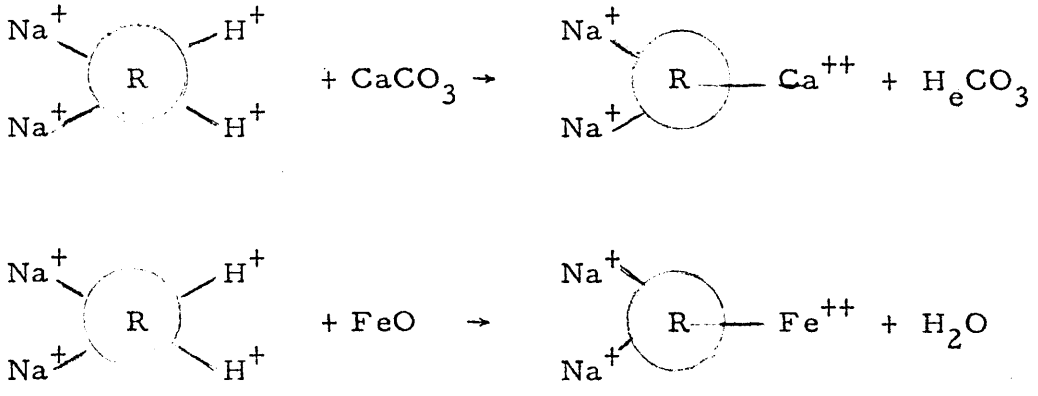
The general test procedure was to take a number of identical samples and to treat them chemically in different manners to study

different factors affecting the soil behaviour. The samples were then subjected to identical consolidation histories with respect to applied stress and stress-time intervals. The test results could then be compared directly and the behavioural differences of the samples could be related to the pre-test sample treatment. The following tests were performed.

Test Ø 3 was made on a sample through which a seepage of artificial pore fluid was maintained. The equivalent NaCl concentration of the pore fluid was 0.3 gm/l. For purposes of determining changes of behaviour of the different samples, this sample was considered as an unaltered sample and was used as a standard for the clay in its naturally-cemented state. Test Ø 7 was a duplication of Test Ø 3 at a later date.

Test 4 was performed on a sample that was permeated with a nearly saturated solution of the sodium salt of ethylene-diamine tetra-acetic acid, or EDTA, a chemical that would remove carbonates ($CaCO_3$, $MgCO_3$), gypsum and iron compounds ; that is, materials that could act as particle bonding agents. Because the soil contained less than 0.1% $CaCO_3$ and gypsum, the use of EDTA would not change the mineralogy of the soil for the short treatment period used in the tests.

The manner in which EDTA operates will be briefly explained. In solution with water it exists as an organic radical R surrounded by H and Na ions. In the presence of $CaCO_3$ and FeO or Fe_2O_3, as examples, it will react by exchanging the H ions for Ca and Fe ions, yielding dissolved complexes containing calcium and iron plus a fluid, and these products can be removed by leaching.



Because of the aggressive nature of EDTA, all metal that could come into contact with it was protected with a surface coating of sintered teflon. Test Ø 8 was a duplication of Test Ø 4 in which the EDTA-treatment period was extended.

Test Ø 5 was performed on a sample leached with a 2 gm/l solution of Calgon. If the electrostatic attraction between negatively-charged surfaces and positively-charged edges of the clay particles in natural soil contributed to the strength of the soil fabric, negating these attraction forces and reversing them to repulsion forces with the use of Calgon would make the structure weaker; that is, more compressible.

Test Ø 6 was performed on a sample that had been permeated with sea water, to investigate the influence of an increased electrolyte concentration in the pore fluid. The salt concentration in the sea water was 27.5 gm equiv. NaCl/l.

Test Procedure and Results. The general test results are summarized in Table 4-3 and Fig. 4-10. Certain test details are given in Figs. 4-11 and 4-12.

A study of Fig. 4-11 will indicate the test procedures that were used. From preliminary tests it was known that the apparent preconsolidation pressure ("break" in the e-log p curve) was about 5 kg/cm^2, and it was decided to make any pore-fluid changes at about $\sigma_a' = 1$ kg/cm^2 at which the soil still possessed considerable reserve strength. Seepage of artificial pore fluid was started at $\sigma_a' \approx 0.6$ kg/cm^2 and pore-fluid changes were made at $\sigma_a' \approx 0.96$ kg/cm^2. Sufficient time was allowed to elapse

before and after any pore-fluid change so that the sample could equilibrate.

There were several factors that governed the length of time allowed for certain of the seepage sequences. In the case of test Ø 4 with EDTA (Fig. 4-11) analyses were made regularly of the effluent from the sample and the flow of EDTA solution was discontinued when the effluent had a specific conductivity equal to about 60% of the specific conductivity of the EDTA solution being introduced into the soil (12 m. mho/cm). (At that time it was feared that EDTA could radically change the mineralogy of the clay and that only a short treatment period could be allowed). The sample was then permeated for over a month with artificial pore fluid until the effluent was nearly clear and the specific conductivity of the effluent was approximately equal to that of the artificial pore fluid. When the test was restarted, 410 cm^3 of pore fluid had passed through the sample, this being greater than 8 times the pore volume of the sample. If an even distribution of flow over the whole sample area had been obtained it is clear that only a fraction of this amount of seepage would have been necessary to remove the EDTA from the sample, but from many similar tests it was known that seepage flow is concentrated through more open zones and that much flow and much time are required to accomplish cleansing by diffusion. One of the most important requirements for tests Ø 3, Ø 4 and Ø 5 was that the pore fluids in all samples were identical upon resumption of loading. Because the EDTA solution was the most drastic pore-fluid change of these three tests, much time was taken to replace it by artificial pore fluid, and because another requirement was that these three samples should remain for similar periods of time under the stress of $\sigma_a' \approx 0.96$ kg/cm^2, tests Ø 3 and Ø 5 were governed by the time taken to remove the EDTA from sample Ø 4.

It will be noticed from Figs. 4-11 and 4-10 that very little compression occurred at $\sigma_a' \approx 0.96$ kg/cm^2 in spite of the fact that pore-fluid changes were made and that over a month's time was allowed to elapse.

After the periods of treatment at $\sigma_a' \approx 0.96$ kg/cm^2, the stress was increased in stress couplets of one short-term load (1 to 2 days) and

one long-term load (5 to 10 days), the load increments decreasing from $\Delta p/p = 0.3$ at the resumption of loading to 0.1 at about P_c' (5 kg/cm^2), and from $\Delta p/p = 0.1$ to 0.06 for stresses greater than P_c'. There were two reasons for this loading procedure. First, it was preferred to use two small $\Delta p/p$ increments rather than one larger value. Second, there was no general tendency for the log-time plot of compression to indicate a marked decrease of rate of secondary compression (Fig. 4-12), and therefore, quite arbitrarily, it was decided to allow every second stress increment to remain for about a week. Although such a procedure is arbitrary, valid test comparisons could be obtained for tests following identical loading schedules. This procedure was successfully followed except for tests Ø 5 and Ø 6 during which loading errors were made at the higher stress levels, and these will be mentioned later.

General information concerning the soils before and after testing are given in Table 4-3. From this table several observations can be made. The initial plasticities and water contents of all the samples were about equal, and the soil was very sensitive. Over the 4-month test period the plasticity of the untreated soil in Ø 3 increased, presumably due to weathering. The plasticities of samples Ø 4 and Ø 5 decreased during the same period, indicating that the EDTA and Calgon treatments of these samples had greater influences than weathering. Sample Ø 6 was much more plastic after the test, indicating the influence of increasing the salt concentration of the pore fluid. The after-test remoulded strengths reflect the changes of plasticity, having increased for plasticity increases and having decreased for plasticity decreases.

In Fig. 4-10 are presented the compression-stress curves for the four tests. For purposes of comparison the curves have been adjusted slightly (maximum adjustment of $\frac{\Delta h}{h_o} = 0.2\%$) in order that all curves began from the same value of $\frac{\Delta h}{h_o}$ at $\sigma_a' = 0.96$ kg/cm^2 after equilibrium. To provide a further comparison, a standard oedometer consolidation test has also been drawn in Fig. 4-10. This test, test F. 82. Ø 4, was performed on untreated natural clay and was loaded at stress intervals of $\Delta p/p = 0.25$ for time intervals of 2 to 3 days.

Tests Ø 3 and Ø 5. From Fig. 4-10 it would appear that the behaviours of tests Ø 3 and Ø 5 are very similar, with the possibility that the Calgon-treated sample (test Ø 5) was slightly more compressible for stresses larger than P_c, and this possibility could be attributed to the influence of Calgon on the edge charges of the clay-mineral particles. However, the differences of compressibility for tests Ø 3 and Ø 5 are small and within the range of sample variation, and to provide another check, test F. 82. Ø 4 was used for comparison. Although this test was not similar in detail to the other tests, the result is so similar to those of tests Ø 3 and Ø 5 that it can be used to show that very little difference exists between tests Ø 3 and Ø 5 and that these differences can fully be explained by natural variations in the soil, even though they might actually be the result of the influence of Calgon. An independent comparison of these tests is given in Fig. 4-10 where a measure of the rate of secondary comparison is plotted against vertical stress, the rate being taken as the slope of the compression-log t curve. For tests Ø 3 and Ø 5 the secondary compressions were measured for the time interval t_1 = 2000 to t_2 = 7000 minutes ($\log t_2 - \log t_1 = \log t_2/t_1 = 0.54$) whereas for test F. 82. Ø 4 in which the load-time intervals were shorter, the secondary compressions were measured for the time interval t_1 = 850 to t_2 = 3000 minutes ($\log t_2 - \log t_1 = 0.54$). Again it is shown that the three samples behaved in much the same manner and it can be concluded that, apparently, Calgon had little influence on the compression behaviour of the soil.

It is of interest to observe when comparing the two diagrams in Fig. 4-10 that the changes of curvature of the compression-log stress curves occur at the same stresses as those for the curves of rate of secondary compression-log stress.

With respect to the secondary-compression results for test Ø 5 in Fig. 4-10, a short explanation will be given for the arrows indicating overestimates and underestimates at stresses 6.46 and 6.96 kg/cm^2 respectively. The correct loading schedule is shown in Fig. 4-11. In test Ø 5, by error the stress σ_a' = 5.96 kg/cm^2 was a short-period increase, and therefore the stress of 6.46 kg/cm^2 followed 2 short-period stress increases rather than one, and this caused the rate of secondary compression at

6.46 kg/cm^2 to be larger than it would have been had it correctly followed only one short-period load increase. The long-period stress of 6.96 kg/cm^2 followed directly the long-period stress of 6.46 kg/cm^2 and therefore the rate of secondary compression is less than it would have been had it followed a short-period load increase. Similar errors were made for test Ø 6 at stresses greater than 7.99 kg/cm^2.

Test Ø 6. Comparison of the test results for tests Ø 6 and Ø 3 in Fig. 4-10 indicates that the introduction of a strong salt-water solution into the soil caused a well-defined increase of the apparent preconsolidation pressure from $P_c' \approx 5$ kg/cm^2 for test Ø 3 to $P_c' \approx 6$ kg/cm^2 for test Ø 6. This decrease of compressibility is due to the depression of the diffuse ionic layers around the clay particles and thereby to a reduction of the repulsion forces between clay particles. The test data also indicate that both the compression curves and the curves of rate of secondary compression for tests Ø 3 and Ø 6 are very similar in form and that the curves for test Ø 6 are displaced with respect to those of test Ø 3 in the directions of larger stress and smaller compressibility.

Test Ø 4. In test Ø 4, by permeating with EDTA, the following cations were removed. The state in which they existed in the natural soil is not known.

Fe^{3+}	3.7 m.gm/gm dry soil
Ca^{2+}	0.6 m.gm/gm dry soil
Mg^{2+}	$\approx$ 0.3 m.gm/gm dry soil

After the test was completed, the original soil and the soil from test Ø 4 were separately thoroughly mixed with the EDTA solution used for the test. The original soil was found to contain 12.5 m.gm Fe^{3+}/gm dry soil and the soil from test Ø 4 contained 10.0 m.gm Fe^{3+}/gm dry soil, the difference being 2.5 m.gm Fe^{3+}/gm. This differs from the value of 3.7 m.gm Fe^{3+}/gm determined from analyses of the leachate, and because this result is the sum of 12 separate determinations on small samples it is believed to be somewhat inaccurate and it is believed that the value of

2.5 m.gm Fe^{3+}/gm dry soil is a more accurate estimate of the amount of iron removed during test Ø 4. The reason why only 20% of the total EDTA-removable iron was removed is thought to be because the permeating EDTA solution followed the more "open" channels in the soil and that the iron in the vicinity of these channels was easily removed, whereas the iron some distance from the channels remained unaffected during this short-term test. In hindsight, the content of available Fe^{+++} (and perhaps Ca^{++}) in the soil should have been determined before the test and compared at regular intervals with the total amounts being removed by the EDTA solution. In this way enough time would have been taken to allow the EDTA solution to diffuse through the soil and to remove considerably more iron and calcium compounds.

In spite of the fact that only a fraction of the available iron compounds and probably only a fraction of the calcium compounds were removed from the soil, their removal had an effect on the compressibility of the soil (Fig. 4-10). By comparing test Ø 4 with test Ø 3 (or test Ø 5) in which the pore fluids were identical, it is clear that the Ø 4 soil was weaker at stresses less than P_c for the untreated soil, and this has been interpreted as indicating that at least some of the iron and calcium compounds that were removed acted in the unaltered samples as bonding agents between the soil particles and strengthened the particle fabric to resist deformations.

Another indication of the change in soil behaviour due to the removal of iron and calcium compounds is the variation with stress of the rate of secondary compression. Secondary compression, believed to be largely a time delay due to a chain-reaction process of particle adjustment, would be exaggerated by bond failures between particles. The removal of bonding materials would cause the break-down of bonds to occur at smaller stresses and this type of behaviour is clearly indicated in Fig. 4-10.

Tests Ø 7 and Ø 8. Because so little of the removable iron was removed by EDTA treatment in test Ø 4, this test was performed again as test Ø 8, with the important difference that the EDTA-treatment period lasted for 38 days in comparison to 13 days for test Ø 4. At the end of

this period the effluent from the sample was unaltered EDTA solution, and this indicated that most of the EDTA-removable iron in the soil had been removed. The test is not yet completed and therefore the exact amount of iron remaining in the clay is not known. The EDTA solution was removed by a seepage flow of 0.3 gm. NaCl/l solution for a period of 34 days, and during this time 208 cm^3 of solution passed through the sample, and at the time loading of the sample was resumed the specific conductivity of the effluent was essentially equal to that of the NaCl solution. Test Ø 7 was performed exactly parallel with test Ø 8 except that a seepage of 0.3 gm NaCl/l was maintained throughout the test, similar to Test Ø 3.

A summary of the test data available to date is given in Fig. 4-10. Test Ø 7 follows exactly test Ø 3, and test Ø 8 shows a great change of compressibility caused by the removal of appreciably more iron than was removed in test Ø 4. In the secondary-compression diagram test Ø 8 plots reasonably close to the other tests up to $\sigma_a' \approx 2$ kg/cm^2, but for the stress $\sigma_a' \approx 3$ kg/cm^2 it is completely off the drawing.

For this Labrador clay there can be no doubt that a considerable proportion of its resistance to deformation is dependent on cementation bonds formed by iron compounds. The removal of these materials has caused a reduction of P_c' by the minimum amount of 2 kg/cm^2 or 20 t/m^2. It will be shown in Chapter 5 that these cementation bonds also have a considerable influence on the shear-strength behaviour of this clay.

Table 4-1. Results of Residual Strength Tests and Characteristic of Minerals and Soils Tested.

Material	Origin	Salinity	Plasticity		Grain Size		Activity	Cation Exchange Capacity	Massive Minerals					Clay Minerals					Residual Strength		
		gm/l	w_L	w_p	Range	<2μ			Quartz	Feldspar	Calcite	Others	Total	Kaolin	Chlorite	Mica and hydrous mica	Mixed layers with Montmor.	Montmorillonite	σ'_n	w_{res}	$\tan\varphi_{res}$
			%	%	μ	%		m.eqv./100 gm	%	%	%	%	%						kg/cm²	%	
Minerals																					
Quartz	Crushed				60-2	0			100										1.0	33	0.69
Quartz	"				< 2	100			100										1.0	-	0.70
Feldspar } both albite	"				60-2	0				100									1.0	30	0.69
Feldspar } and microcline	"				< 2	100				100									1.0		0.59
Calcite (crushed)	"				60-2	0					100								1.0	22	0.6
Sahara sand	Dune				>200	0													1.0	33	0.58
SiO_2 (amorphous)	Flue gas		33	28	0.5-0.01	100	0.05	2											1.0	35	0.56
Muscovite	Crushed	0			60-20	0										100			1.0	97	0.36
Muscovite	"	0	63	55	< 2	100	0.08	28											1.0	70+	0.45
Na-muscovite	"	30	98	62	< 2	100	0.36	28								↓			1.0		0.33
Na-muscovite	"	0			< 2	100		28								100			1.0	-	0.31
Na-hydrous mica	Fine fraction of Selnes quick clay	30	99	39	< 2	100	0.60	39.5		5		5	10		5	85			1.0	46	0.43
Na-hydrous mica		0	51	33	< 2	100	0.18												1.0	39	0.29
Na-hydrous mica + Calgon		5 Calgon	38	30	< 2	100	0.08												1.0		0.30
Na-hydrous mica + Calgon		0	-	-	< 2	100	-												1.0	70	0.27
K-hydrous mica		30	118	46	< 2	100	0.72												1.0	47	0.46
K-hydrous mica		0	84	39	< 2	100	0.45												1.0	45	0.39
Ca-hydrous mica		15	-		< 2	100	-	36.0											1.0	41	0.47
Ca-hydrous mica		0	-	-	< 2	100	-	36.0											1.0	42	0.44
Na-hydrous mica		30	151	53	< 0.5	100	0.98	28											1.0	51+	0.49
Na-hydrous mica		0	87	46	< 0.5	100	0.41	28		5		5	10		5	85			1.0	-	0.37
Na-montmorillonite	Fine fraction of K-W bentonite	30	620	45	< 2	100	5.8	115										100	1.0	220	0.18
Na-montmorillonite		0	1325	53	< 2	100	12.7	115										100	1.0	465	0.07
Na-montmorillonite		30	995	47	< 0.1	100	9.5	-										100	1.0	160	0.15
Na-montmorillonite		0	1995	56	< 0.1	100	19.4	-										100	1.0	445	0.07
Na-montmor. + Calgon		0	2165	57	< 0.1	100	21.1	-										100	1.0	475	0.05
Ca-montmorillonite		0	795	47	< 0.1	100	7.5	-										100	1.0	135	0.17
Soils.																					
Labrador, Canada	Quick clays	< 2	32	22	< 60	55	0.18	13	10	20			30		15	55			1.0	26	0.52
Selnes, Norway		< 2	23	17	< 60	42	0.16	11	25	10		15	50		5	45			1.0	21	0.60
Manglerud III, Norway		< 2	34	21	< 60	44	0.30	11	15	15	1	5	35		15	45	5		1.0	25	0.56
Åsrum I, Norway		< 2	37	22	< 60	43	0.34	9	15	15		10	40		15	45			1.0	25	0.50
Skøyen, Norway	Insensitive clays	27	41	23	< 60	47	0.38	7	20	30	1		50		15	35			-	-	-
Ottawa, Canada		< 2	67	31	< 60	70	0.52	20	10	20		10	40		5	50	5		1.0	38	0.55
Sandnes I, Norway		< 2	31	18	< 200	42	0.30	13	20	10	5	5	40		20	30	10		1.0	24	0.53
Sandnes II, Norway		< 2	71	28	< 60	80	0.55	-		10	5	10	25	5	15	40	10	5	1.0	52	0.25
Little Belt, Denmark	Clay shales	11	140	46	< 60	58	1.6	45	2	+	17	5	25	10		10	5	50	1.0	68	0.16
Bearpaw, Canada		-	117	35	< 60	50	1.6	25	20	5	1		25			15	60		1.0	69	0.11
Pierre, USA			145	42	< 20	56	1.8	39	15	5			20			10	← 70 →		1.0	78	0.10
Pepper, USA			96	25	< 60	58	1.2	25	15	10	0.2	5	30	45		15	5	5	1.0	58	0.10
Cucaracha, Panama			59	32	< 100	(36)	0.7	61	10				10	←		90	→		1.0	42	0.11
Vajont I, Italy			106	35	< 75	70	1.0	55	5	5	30	10	50				← 50 →		1.0	-	0.18
Vajont II, Italy			106	40	< 75	68	1.0	79	10		7	5	20			5	← 75 →		1.0	18	0.16
Vajont III, Italy		-	62	18	< 60	52	0.8	27	5		40		45			30	← 25 →		1.0	27	0.28
Treated Soils																					
Na-Grundite, USA.		30	130	43	< 2	100	0.85	26								70	30		1.0	69	0.18
Na-Grundite		0	164	46	< 2	100	1.2	26											1.0	68	0.20
Na-Grundite + Calgon			139	39	< 2	100	1.0	26											1.0	-	0.06
Ca-Grundite		0	163	49	< 2	100	1.2	26								70	30		1.0	80	0.18

TABLE 4-2. Information Concerning Samples Used in Leaching Tests.

Test No. (F.266)	Field Data			Laboratory Pressures				Strength			Water Content						Remarks
	Hole no.	Depth m	P_o' kg/cm^2	P_c' kg/cm^2	P_{leach}	$\frac{P_l}{P_o'}$	$\frac{P_l}{P_c'}$	s_u t/m^2	$s_{\bar{u}}$ t/m^2	St	w %	w_L %	w_p %	I_p %	I_L	Salt Content gm/l	
Manglerud III																	
Ø 6	4	7.6	0.50	0.55	1.15	2.3	2.1	1.8	0.02	90	47.0	29.3	22.0	7.3	3.4	≈ 2	At a salt content of 30 gm/l. this clay had the following plasticity limits: w_L = 44 to 47% w_p = 21 to 25%
								-	-	-	32.2	21.7	17.3	4.4	3.4	1.2	
Ø 7	5	4.9	0.27	0.35	0.35	1.3	1.0	1.1	0.04	28	44.6	32.2	21.4	10.8	2.15	1.5	
								0.67	<0.01	>67	38.2	ca 23	18.5	ca 4	> 3	0.9	
Ø 9	5	4.8	0.27	0.35	0.45	1.7	1.3	1.0	0.04	25	44.1	30.9	19.6	11.3	2.17	1.6	
								0.52	<0.01	> 52	32.5	ca 23	18.6	ca. 4	> 3	2.5	
Sköyen																	
Ø 121	1	22.3	1.84	0.8 P_o'	1.13	0.62	0.8	4.0	0.92	4.3	29.2	40.8	19.8	21.0	0.45	26.5	
								6.2	1.64	3.8	29.0	32.9	21.2	11.7	0.67	1.0	
Ø 122	1	22.4	1.84	"	1.46	0.79	0.9	4.0	0.92	4.3	38.2	46.1	24.2	21.9	0.64	26.6	
								3.5	0.04	87	36.6	30.4	23.6	6.8	1.91	0.9	
Ø 123	1	22.3	1.83	"	1.65	0.90	1.1	4.0	0.92	4.3	35.2	46.1	22.6	23.5	0.54	27.2	
								5.8	1.25	4.6	32.8	33.0	24.5	8.5	0.98	1.1	
Ø 125	1	22.5	1.85	"	2.31	1.25	1.6	4.0	0.86	4.7	39.1	46.7	23.4	23.3	0.67	27.1	
								4.3	0.42	10	32.0	31.2	24.2	7.0	1.11	1.0	
Ø 126	1	22.2	1.83	"	1.86	1.02	1.3	3.9	0.89	4.4	39.2	53.4	24.5	28.9	0.51	27.4	
								7.4	2.3	3.3	33.6	48.6	26.1	22.5	0.33	30.1	

Note. For all tests, the first line contains before-test results, the second line contains after-test results.

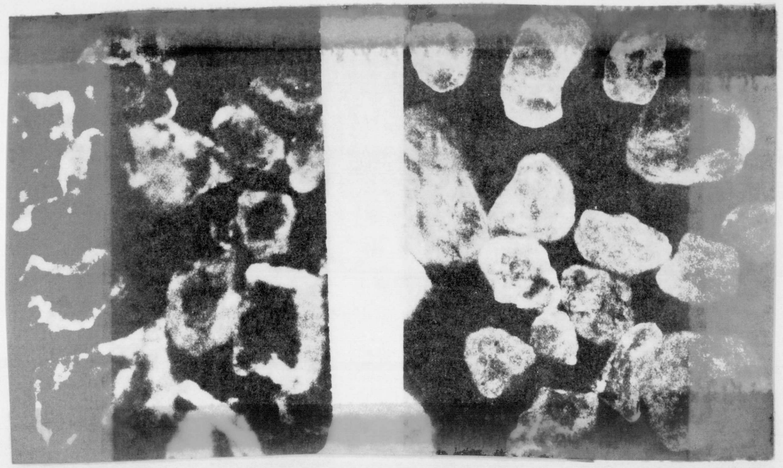
Table 4-2.

TABLE 4-3. Properties of individual samples of Labrador clay before and after compressibility tests.

Test	Strength			Water Content					Salt Content gm/l	Date Started 1966	Test Duration days	Treatment
	s_u t/m^2	$s_{\tilde{u}}$ t/m^2	St	w %	w_L %	w_p %	I_p	I_L				
Ø 3	5.8	0.05	115	35.2	30.3	20.5			2.2	13/1	138	None
	4.0	0.32	13	33.8	34.8	18.5			0.6			
Ø 4	5.3	0.03	180	37.0	31.9	22.2			2.0	13/1	138	EDTA
	1.9	0.06	32	37.2	29.3	20.0			0.4			
Ø 5	-	-	-	35.6	31.6	21.6			0.7	23/2	128	Calgon
	3.6	0.12	30	33.4	29.0	20.2			1.4			
Ø 6	-	-	-	36.7	33.2	22.9			1.4	23/2	116	NaCl
	6.7	2.0	3	35.7	42.2	23.8			34.0		Not com-	
Ø 7	6.7	0.21	30	36.2	33.7	23.8			-	26/8	pleted.	None
Ø 8	8.3	0.25	33	35.2	30.8	22.1			-	26/8		EDTA
F. 82-Ø 4	5.1	0.21	24	36.9	32.0	24.2			1.3		75	None
	5.6	0.46	12	32.2	33.1	20.4			-			

Note: For all tests, the first line contains before-test results, the second line contains after-test results.

Table 4-3.

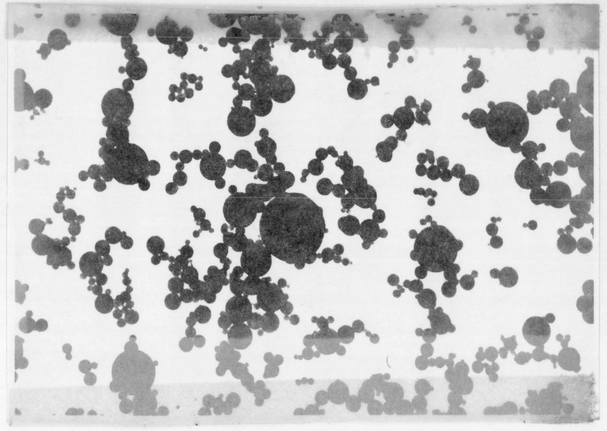


0 100 200 μ
0 0.1 0.2 mm

SILT FRACTION OF A TILL

0 100 200 300 μ
0 0.1 0.2 0.3 mm

SAHARA DESERT DUNE SAND



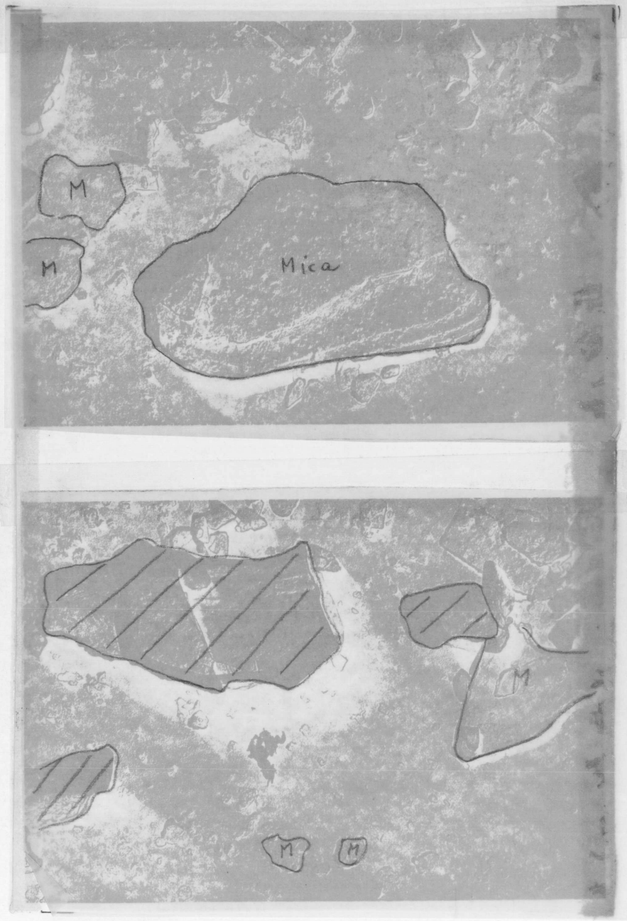
0 1 μ

SILICA DUST FROM A SMELTER CHIMNEY



0 2 4 μ

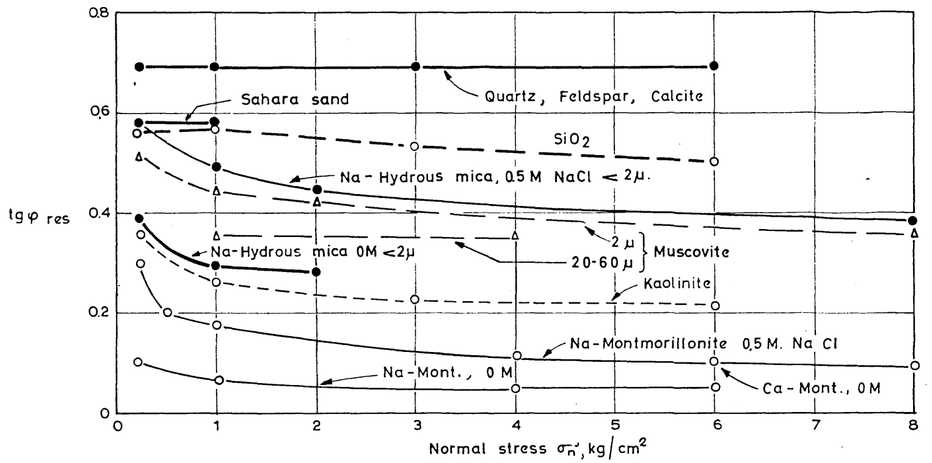
CLAY FRACTION OF SELNES QUICK CLAY

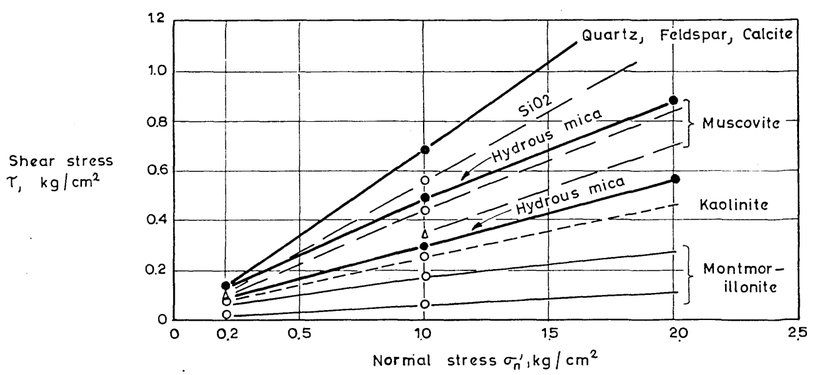


0 1 2 μ

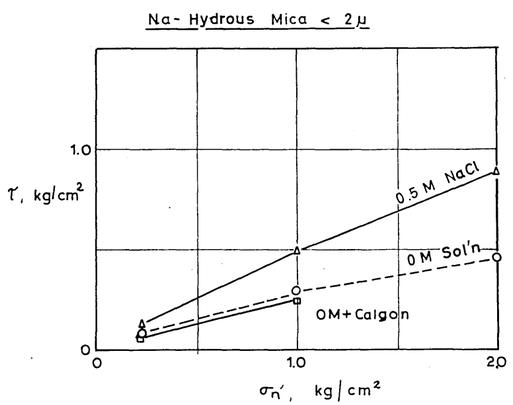
CLAY FRACTION OF SELNES QUICK CLAY

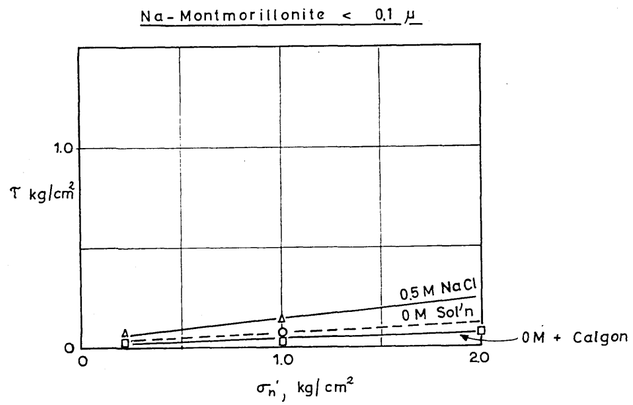
PHOTOGRAPHS OF SiO_2 AND MINERAL COMPONENTS OF NATURAL SOIL

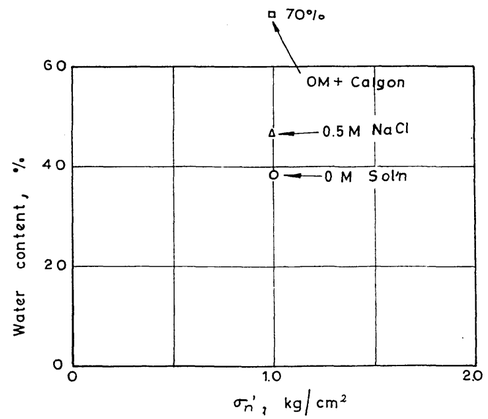


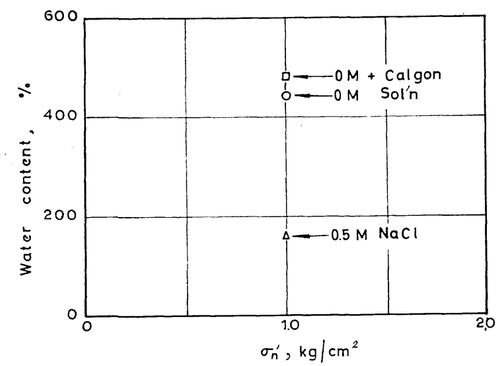


A. SHEAR TESTS ON MINERALS





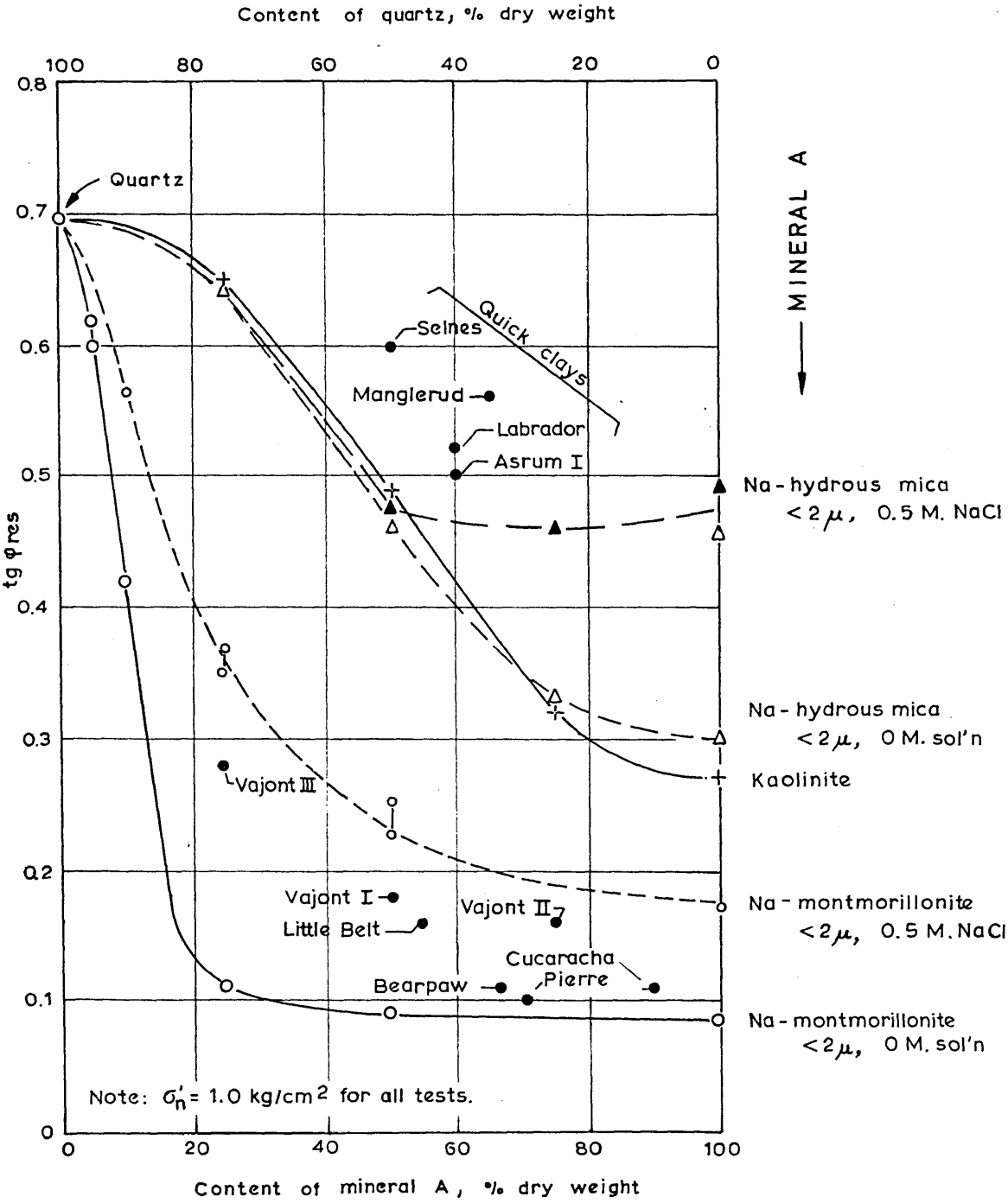




B. TYPICAL TEST RESULTS

RESIDUAL SHEAR STRENGTH OF FINE-GRAINED MINERALS

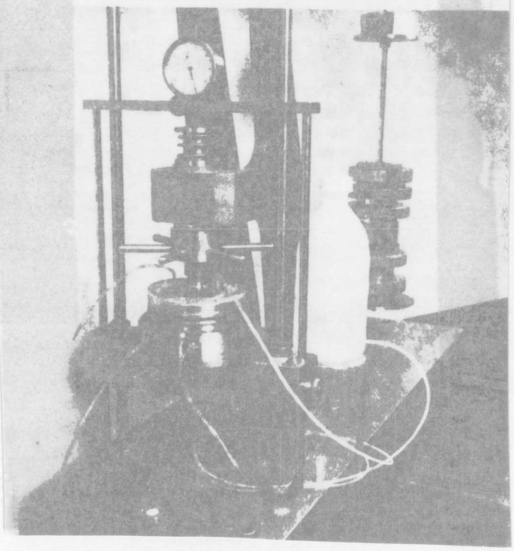
Fig. 4-3



RESIDUAL STRENGTHS OF MINERAL MIXTURES AND NATURAL SOILS

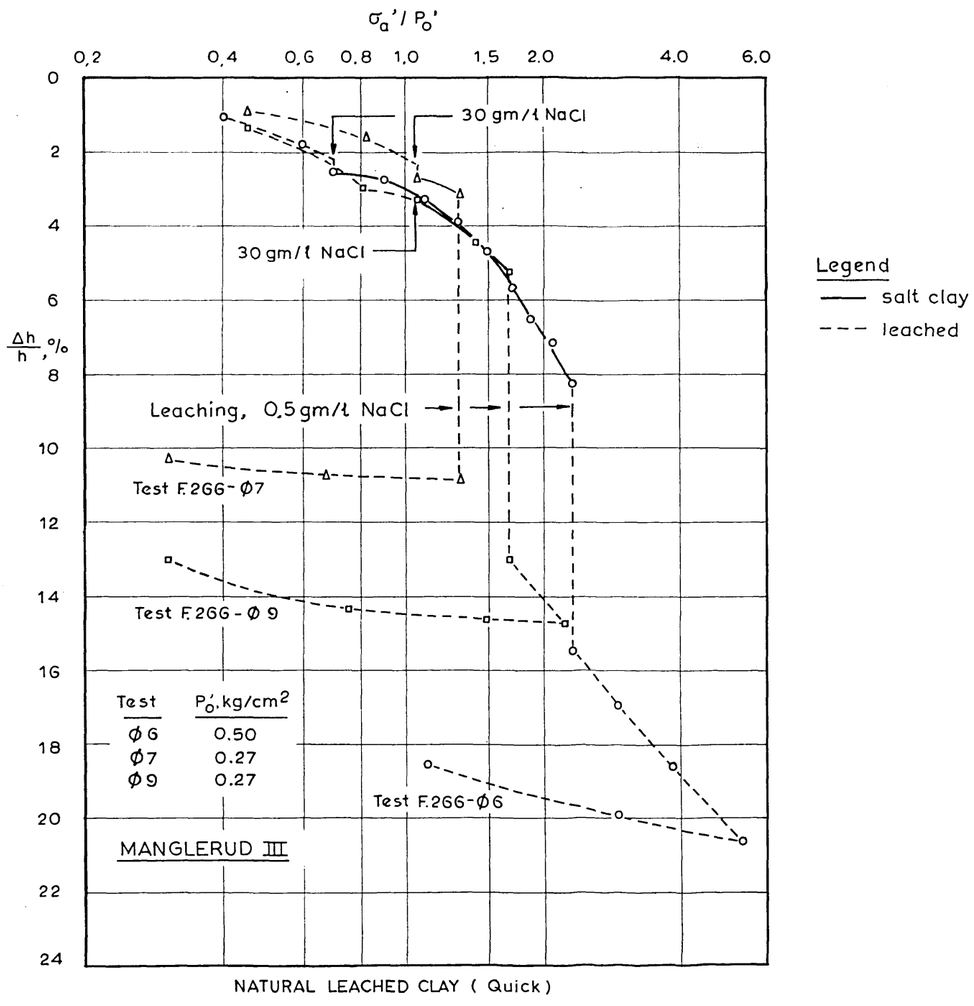


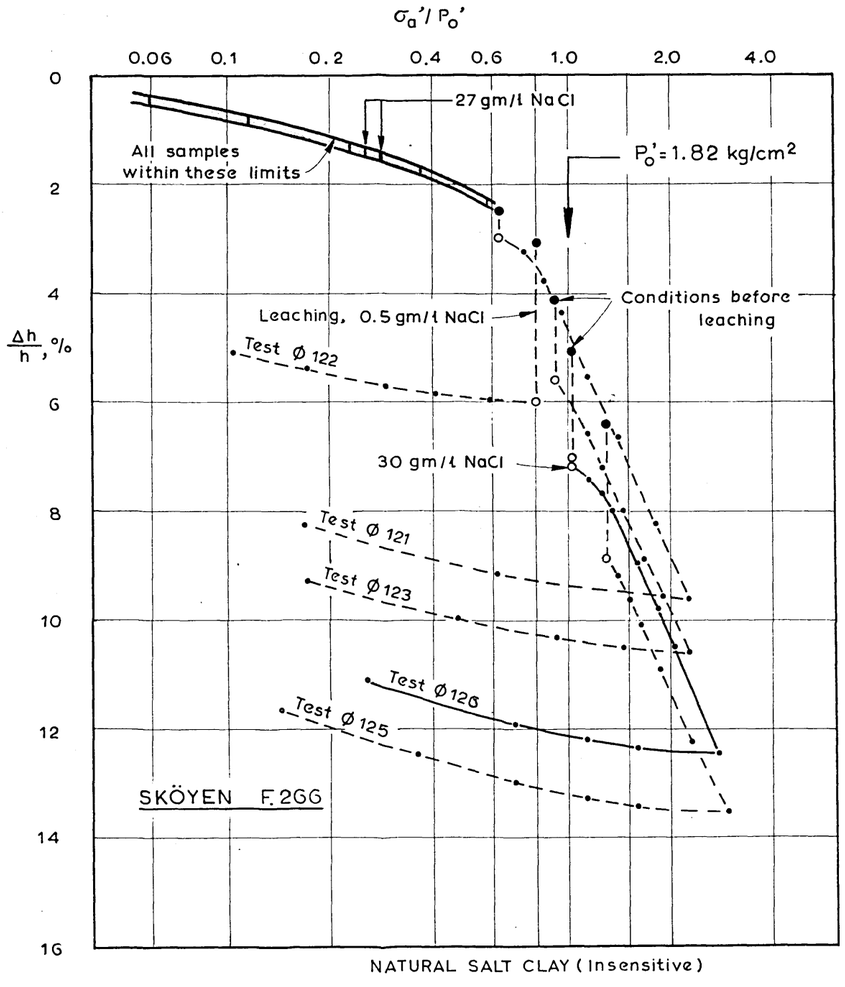
TEST ARRANGEMENT



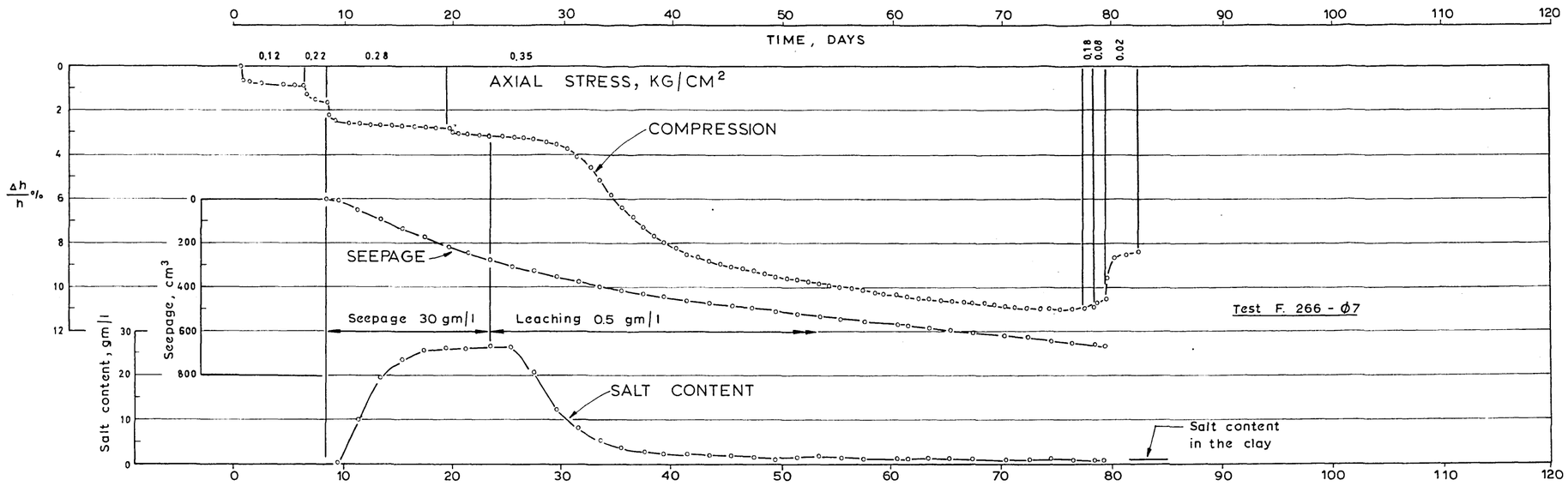
OEDOMETER CELL

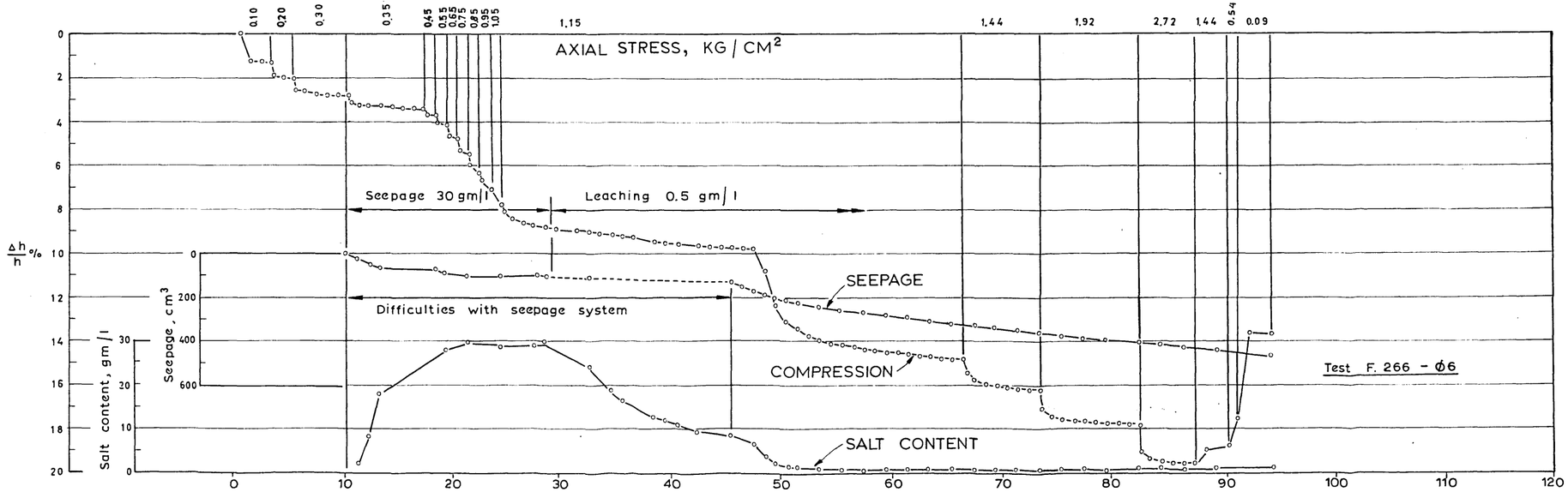
PHOTOGRAPHS OF APPARATUS USED FOR OEDOMETER - LEACHING



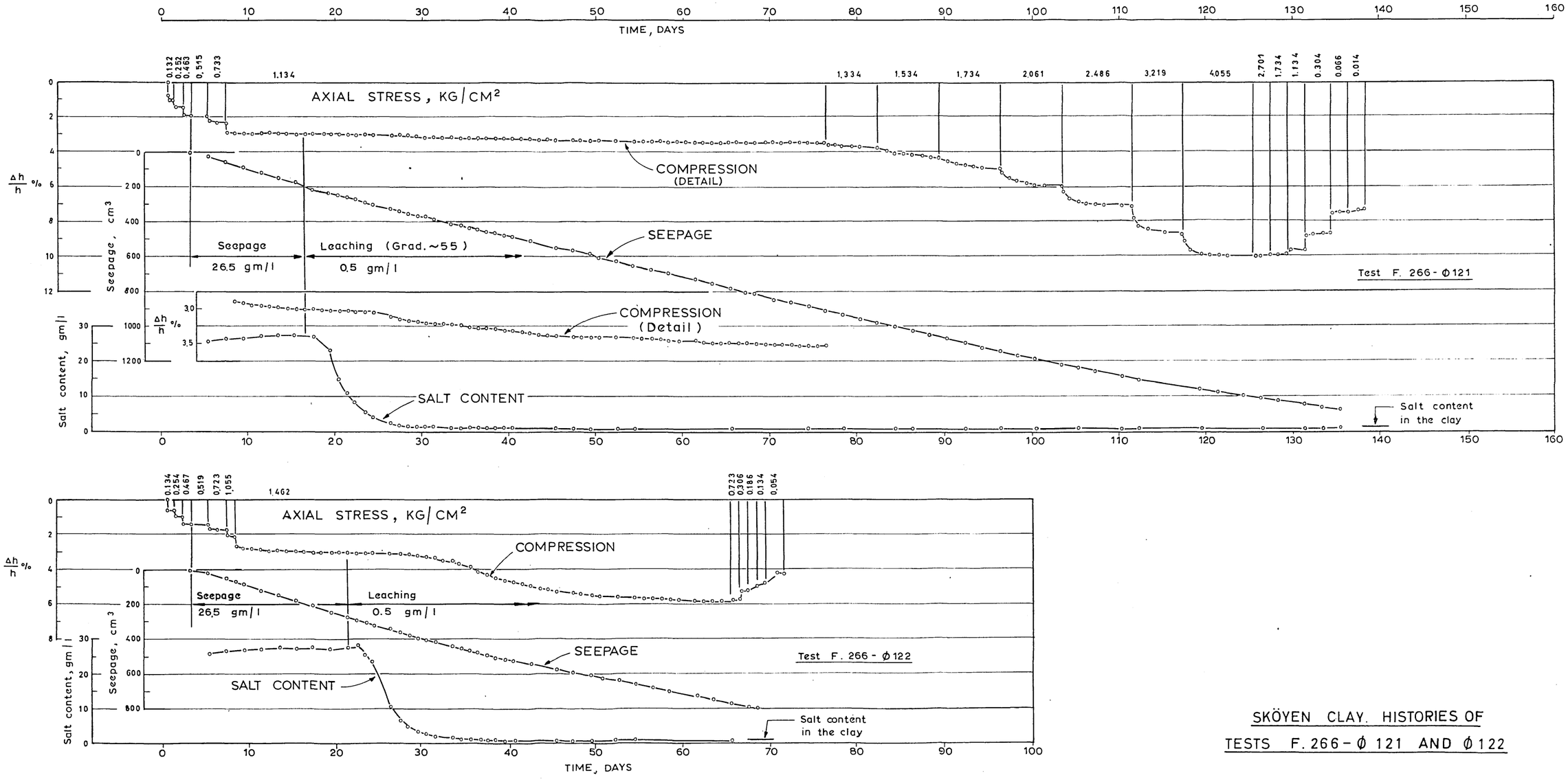


EFFECTS OF CHANGING THE PORE - FLUID SALT CONCENTRATIONS OF TWO UNDISTURBED CLAYS



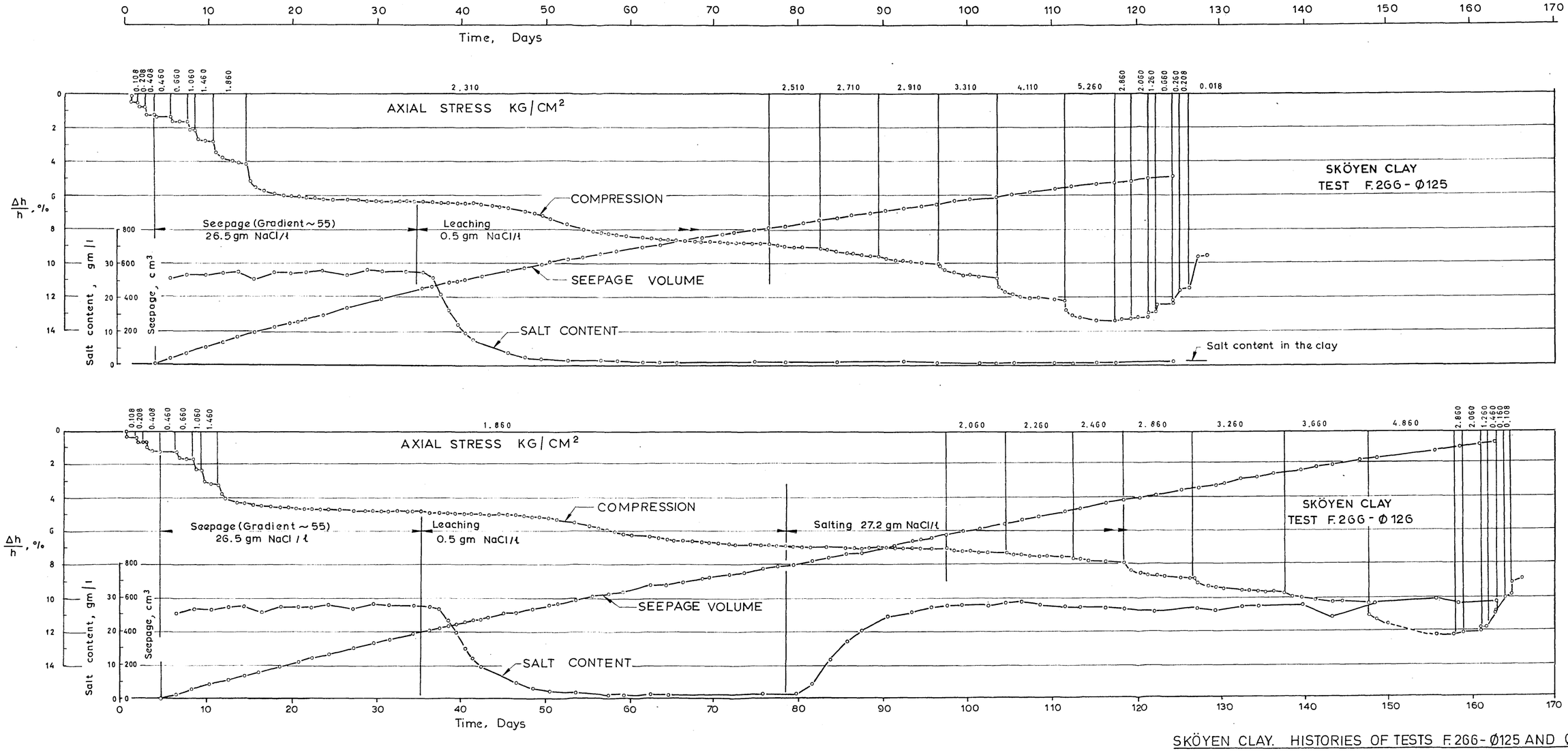


MANGLERUD III CLAY. HISTORIES OF TESTS F. 266 - ϕ6 AND ϕ7

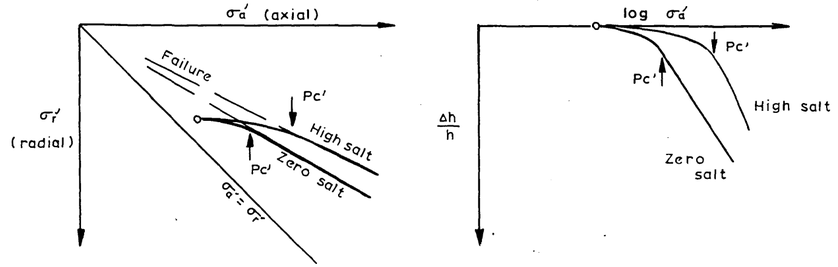


SKÖYEN CLAY. HISTORIES OF TESTS F. 266-Ø 121 AND Ø 122

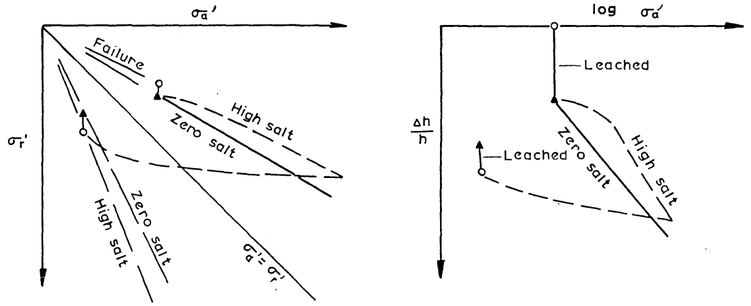
Fig. 4-8



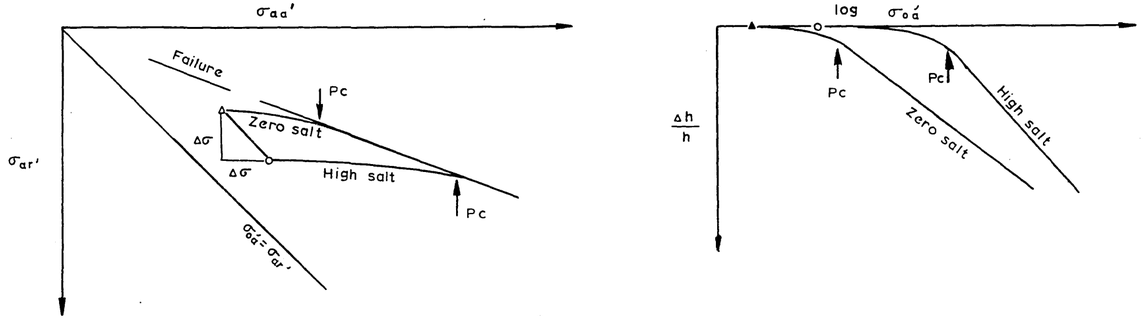
SKÖYEN CLAY. HISTORIES OF TESTS F.266-Ø125 AND Ø126



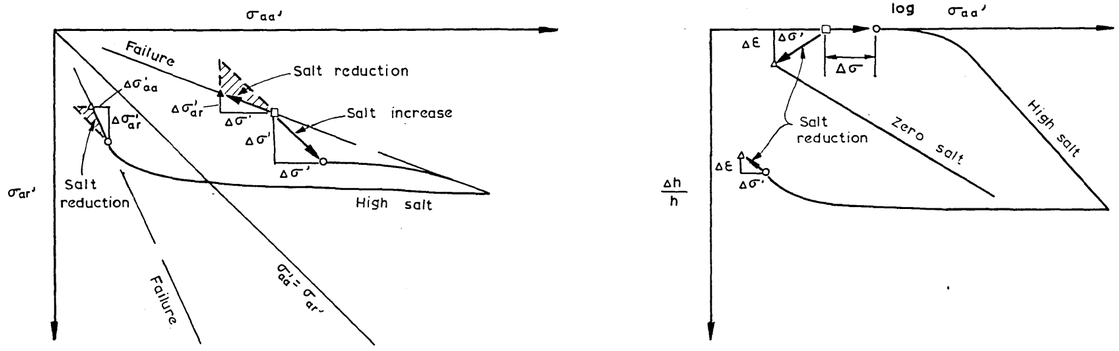
A. CHANGES OF SALT CONTENT AT σ_a'/σ_r' < FAILURE



B. CHANGES OF SALT CONTENT AT σ_a'/σ_r' = FAILURE



C. CHANGES OF SALT CONTENT AT $\sigma_{aa}'/\sigma_{ar}'$ <FAILURE

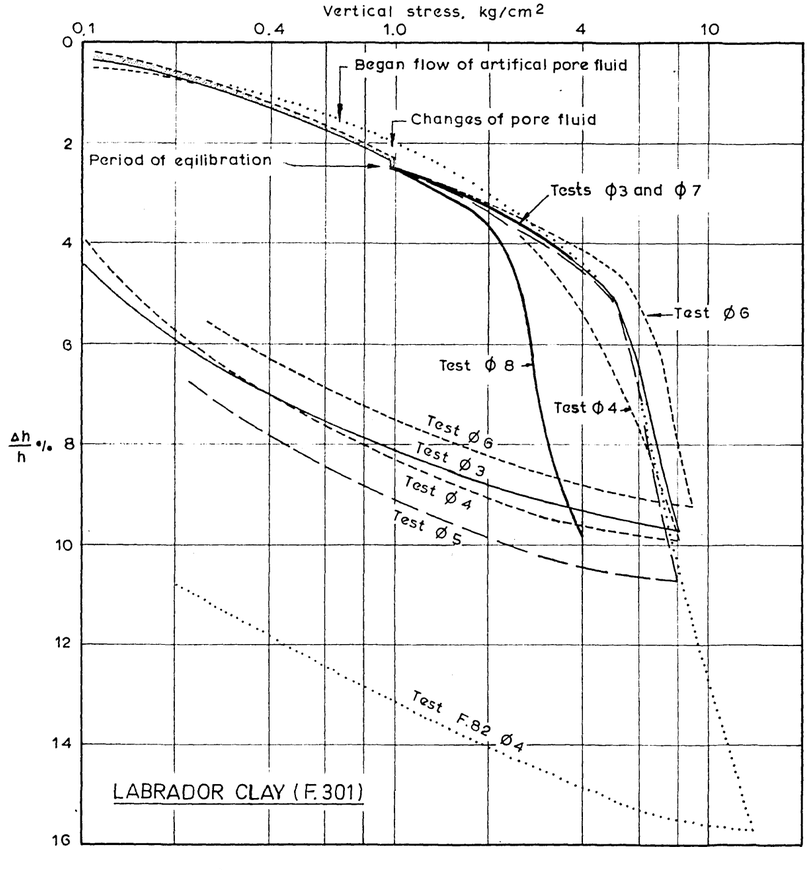


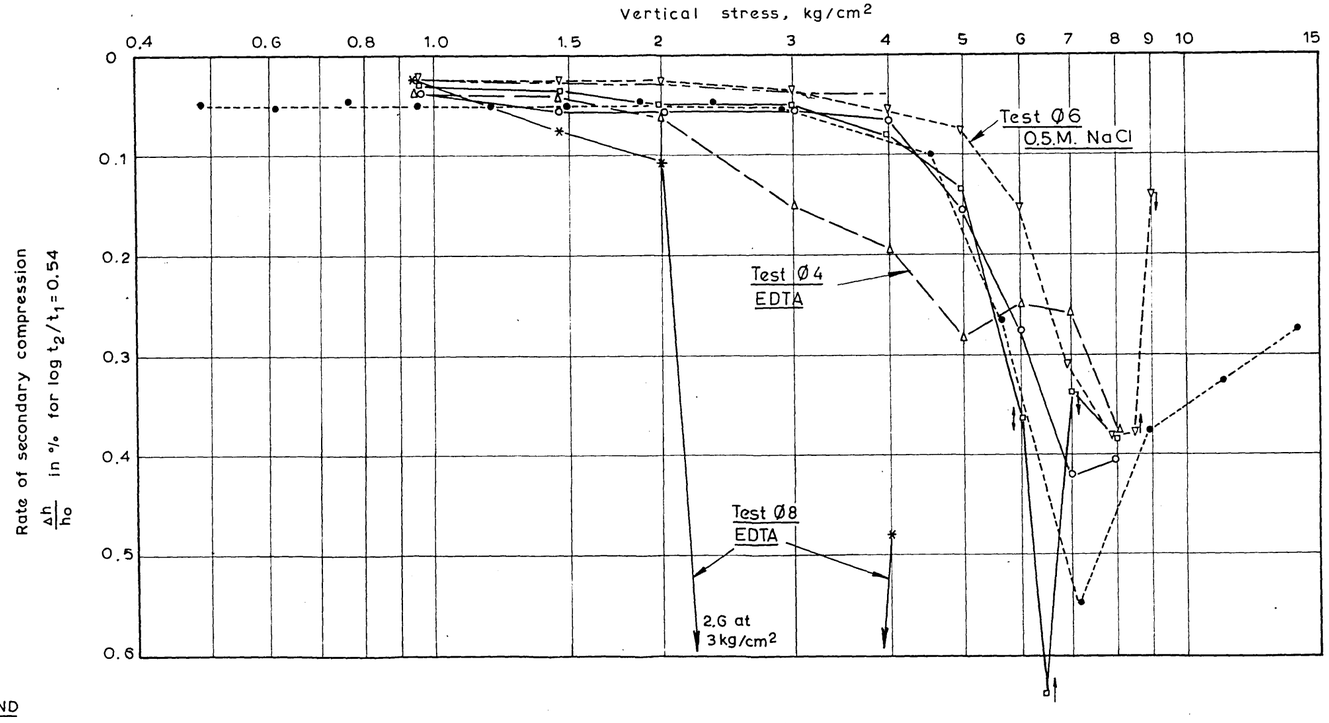
D. CHANGES OF SALT CONTENT AT $\sigma_{aa}'/\sigma_{ar}'$ = FAILURE

METHOD 1. SHEAR RESISTANCE DEPENDENT ON SALT CONCENTRATION

METHOD 2. ABSOLUTE STRESS AT PARTICLE CONTACT DEPENDENT ON SALT CONCENTRATION

EFFECTS ON COMPRESSIBILITY OF CHANGES OF ELECTROLYTE CONCENTRATION





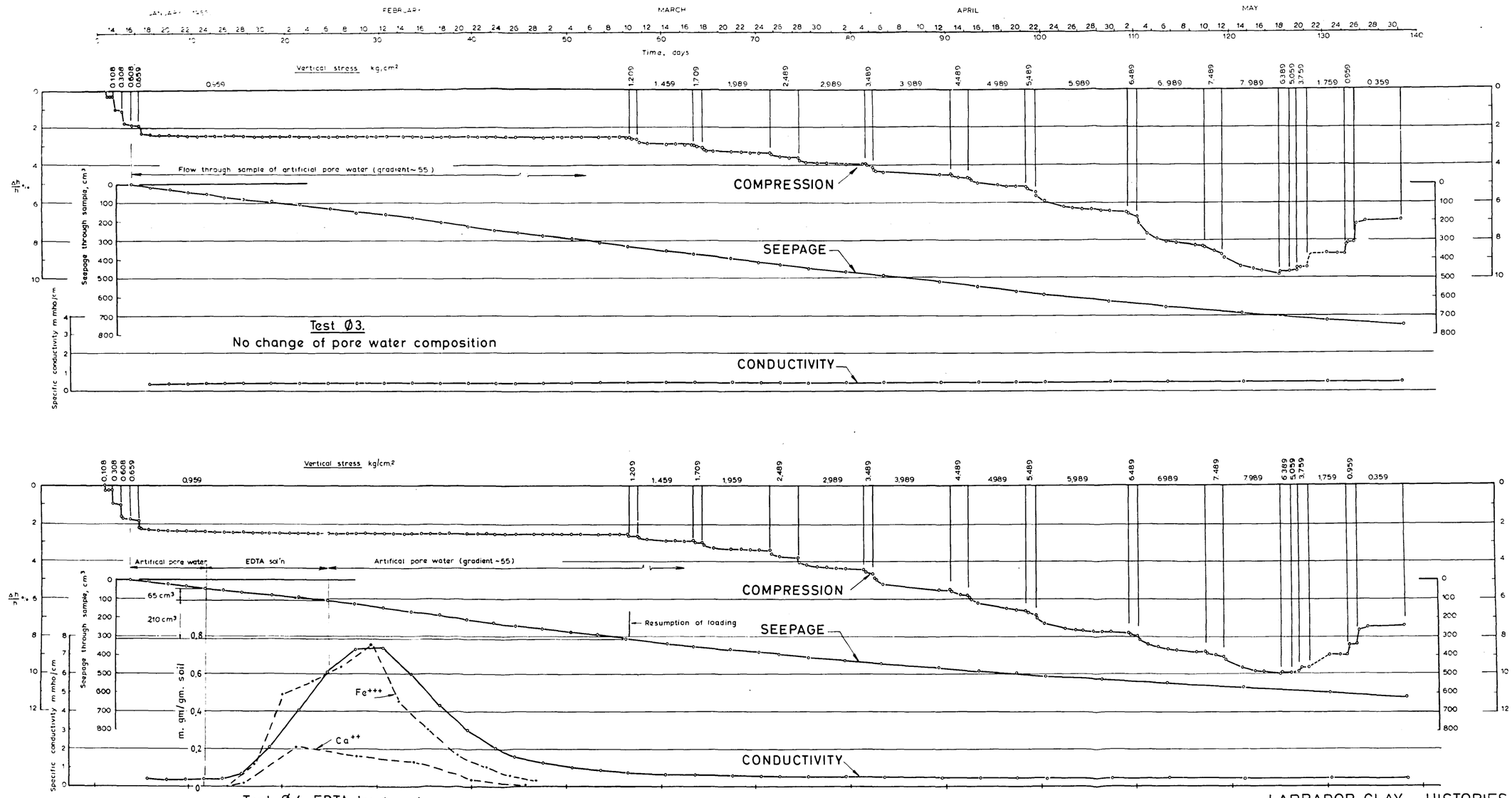
LEGEND

- Test Ø3 - no change
- Test Ø4 - EDTA
- Test Ø5 - Calgon
- Test Ø6 - 0.5 M. NaCl
- Test Ø7 - no change
- Test Ø8 - EDTA

t_1 = 2000 min.
t_2 = 7000 min.

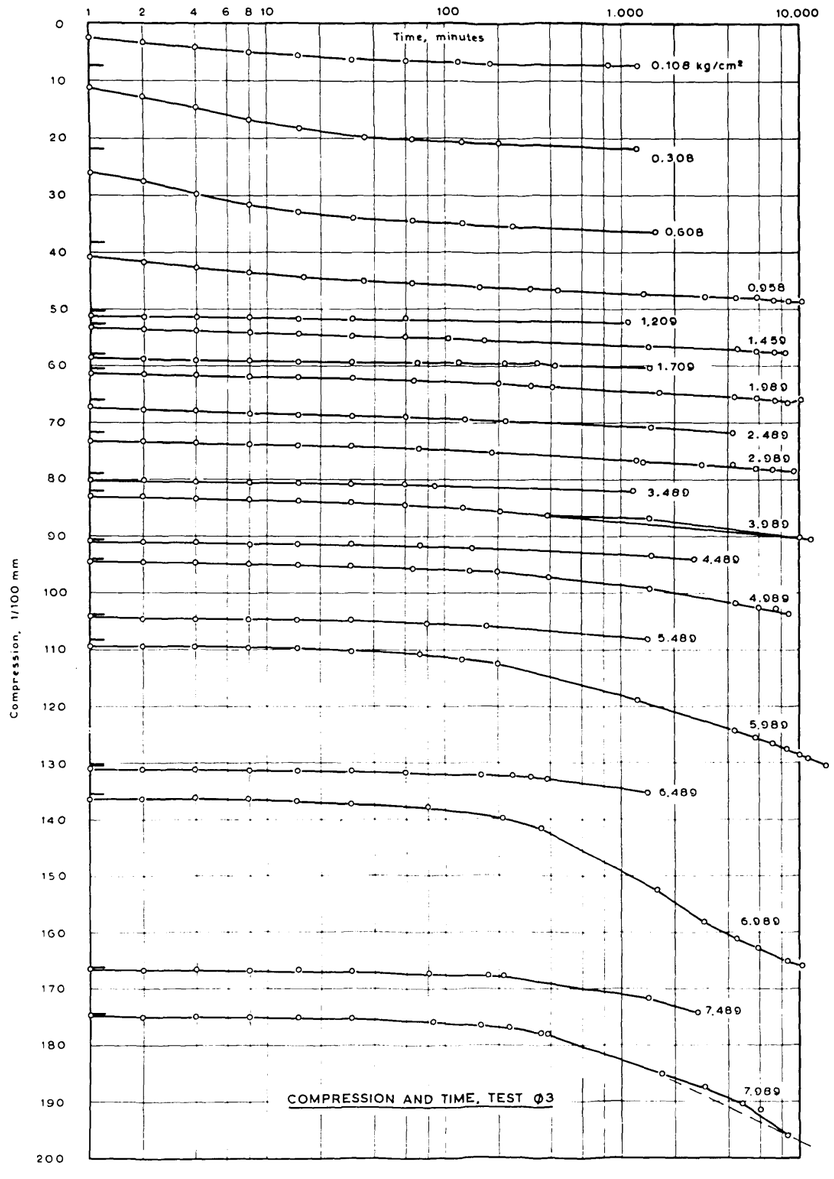
- Test F.82 - Ø4 - no change
  t_1 = 850 min.
  t_2 = 3000 min.

EFFECTS ON THE COMPRESSION BEHAVIOUR OF A CEMENTED QUICK CLAY OF REMOVING IRON COMPOUNDS AND CHANGING PORE-FLUID COMPOSITION

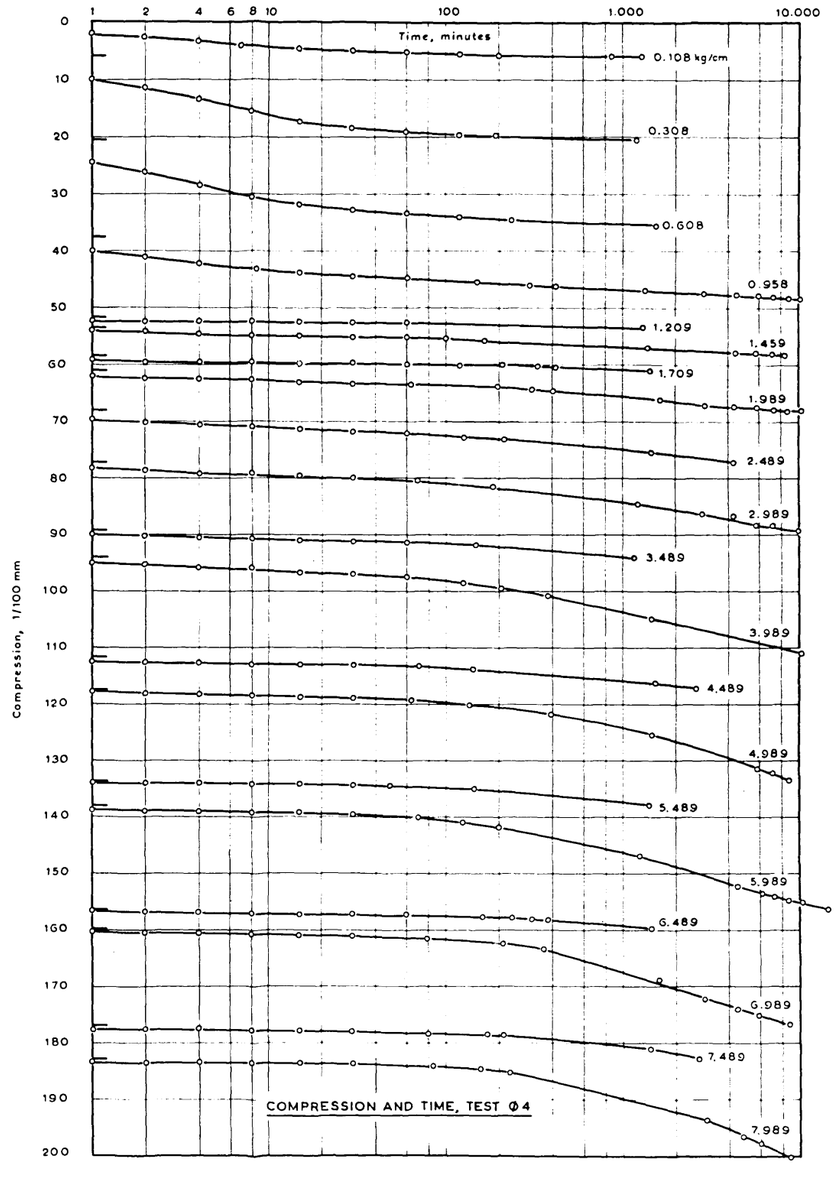


Test. Ø 4 EDTA treatment

LABRADOR CLAY. HISTORIES OF TESTS.
F. 301-Ø3 and Ø4.



Note: Initial sample thicknesses were 20 mm.



COMPRESSION - TIME RESULTS OF TESTS Ø3 AND Ø4 (F. 301)

Chapter 5. Shear-Strength Behaviour of Natural Quick Clays.

1. Introduction.

In the foregoing chapters results have been presented of K_o-measurements, strength determinations for slide investigations, and investigations of some physico-chemical properties of soils. In this chapter these results plus the results of laboratory strength tests on quick clays will be brought together for the purpose of interpreting the observed behaviours. In the light of what we know concerning the possible existence of brittle bonds as a strength component, consideration will be given to K_o and what values might be expected for different natural conditions. The behaviour of samples in triaxial and direct simple-shear will be considered in relation to the problem of strength measurement for purposes of engineering design. The results of some investigations of the problem of sample disturbance in quick clays will be presented in Appendix 3.

The primary purpose of this chapter is to attempt to explain how quick clay will behave under different test or field conditions. This explanation will be made in general terms of the strength components bonds and friction, principal effective stresses, and strain. No attempt will be made to go deeply into the fundamental aspects of shear strength as has been done by Hvorslev (1960) and Schmertmann (1964) or into the effects of dilatancy (Rowe, Oates and Skermer, 1964).

2. Components of Shear Resistance.

"Friction". The results of the shear tests on pure minerals and on remoulded natural soils indicated that the residual shear resistances of these uncemented materials were essentially directly proportional to applied effective stress (Fig. 4-2), and as explained previously, these strength values are measures of the maximum strengths that can be expected for very loose structured, uncemented soils. Because such soils are sensitive to stress increases, large volume decreases occur

during shear and considerable strain is required to fully mobilize the maximum effective-stress shear resistance of the soil, and this is clearly indicated in shear tests on quick clays in which consolidation pressures have been used that are sufficiently large to destroy cementation bonds (see test 11, Fig. 5-5).

This strength component, that is nearly directly proportional to applied effective stress, will be called friction for convenience, although it is realized that some of this resistance is not necessarily dependent only on applied stress. In non-plastic Norwegian clays the Hvorslev-type cohesion (Hvorslev, 1960) is very small and the soils are very nearly truly frictional (Sevaldson, 1956; Simons, 1960 a, 1960b).

Cementation Bonds. The other strength component that will be considered is cementation bonds (and weld bonds) between individual particles. Unfortunately, there is little known concerning the nature of such bonds, except in the case of Labrador clay where iron compounds were found to be important, but even in this case the actual chemical and physical nature of the bonds is not known. Therefore, the term cementation bonds is meant to be a general term that includes all brittle (strain sensitive) strength components, such as those arising from the cementing action of chemical precipitates or from particle welding by recrystallization. To the author's knowledge some of the first measurements of their influence on the strength of sensitive clays were made by A. Casagrande in about 1930 using beam tests (A. Casagrande, private communication), and in later years there has been the work of Bjerrum and Wu (1960) with compression tests, Crawford (1963) with drained unconfined compression tests, and Conlon (1966) with tension tests. From this work, and that to be later described, it has been found that the cementation-type bonds are brittle and fail at very small strains.

The variation in magnitude of the strength contribution by these bonds in different soils is shown in Fig. 5-1; the diagrams on the left side give the full range of stresses used in the laboratory tests,

and on the right side the stress range is limited to that generally encountered in engineering practice. It is clear that the Manglerud clay from Norway has very little cementation-bond strength, and this result is similar to those obtained for the Selnes and Ullensaker quick clays (Figs. 3-4 and 3-7). That these clays possess bond strength at all is known from the fact that they exist in normally-consolidated states at water contents well in excess of their liquid limits, they exhibit strengths in drained unconfined compression tests and they do not slake when placed in water solutions having the same compositions as the natural pore fluids. The Ottawa and Toulnustanc clays are examples of highly bonded clays.

Effect of Rate of Strain. Rate of strain is a factor that is frequently mentioned with respect to shear strength, a factor that has been shown to have great influence on the behaviour of soils tested in undrained compression (Taylor, 1943; Casagrande and Wilson, 1951; Bjerrum, Simons and Torblaa, 1958; Crawford, 1963), and yet a factor whose influence on the more basic soil properties of friction and cementation-bond has been left uninvestigated. The experiments to be described are limited in number and in scope but they indicate that rate of strain does not have great influence on the magnitude of friction and that its influence on cementation bonds might vary considerably for different soils, due, possibly, to the reformation of broken bonds.

In Fig. 5-2 are presented the results of rate-of-strain tests on uncemented clays. The results in diagram A are typical for samples of pure minerals and remoulded clays that were tested in their residual-strength state. Some samples were sheared at a number of different specific rates of strain (see test on kaolinite, Ca-montmorillonite, and Na-montmorillonite OM. $< 0.1\mu$). In other tests, samples were brought to their residual strength and then the motor of the direct-shear apparatus was stopped and the sample was allowed to strain under the influence of the stressed load gauge (proving ring); the rate of creep of the sample at different stages of the test was determined from the slope of the deformation-time curve, and $\text{tg}\,\varphi_{res}$ was obtained from the load gauge

measurements. All tests were fully drained and the results showed that the residual shear resistance of uncemented soils was very little affected by strain rate within the range investigated. In diagram B are presented the results of triaxial compression tests on a natural insensitive Norwegian clay (Bjerrum, Simons and Torblaa, 1958) that can be considered as an uncemented clay due to the fact that the samples during pre-test consolidation experienced volume decreases usually greater than 10%, for which all cementation-type bonds would have been destroyed. The samples were deformed at different rates of strain, either in drained tests or in undrained tests with pore pressure measurements. Although there is scatter of the results, it appears that strain rate has little influence on the frictional behaviour of this clay when measured with respect to effective stresses.

No other test results could be found concerning the influence of rate of strain on the frictional resistance of non-cemented soils.

In Fig. 5-3 and Table 5-1 are presented the results of some drained compression tests on samples of the cemented Labrador clay. The tests were performed at a small cell pressure (see diagram of compressibility characteristics) in order that the bond-strength contribution to the strength of the samples was very much larger than the frictional-strength contribution, and therefore that any changes of sample behaviour resulting from changes of strain rate could be associated with the influence of strain rate on bond strength. The purpose of the tests was to investigate the possibility that bond strength was sensitive to strain rate or time; it could be imagined that cementation bonds were time dependent, and that if more time was allowed, more bonds would break, and therefore the potential strength of the soil would decrease with time.

To ensure that the cell pressures in all tests remained constant and equal, the five triaxial cells were connected to a common mercury-pot cell-pressure system. Tests 16, 17 and 19 were strained at different constant rates of deformation. Test 18 was loaded in stages

and between each stage the sample was allowed to strain under constant axial load. Test 15 was performed to investigate the possibility that weathering and/or aging (Bjerrum and Lo, 1963) during the longer tests could influence the results. The sample was consolidated and allowed to age under this cell pressure for a period of 80 days, and then it was strained to failure at the same strain rate that was used in test 16.

Only five tests were performed and this number is too few to account for natural variations of the soil. Therefore, the following conclusions are tentative, in spite of the fact that the test results appear to be regular.

1. Tests 15 and 16, strained at the same rate, indicated that laboratory-age differences had no effect on sample strength of this soil.

2. All samples failed at equal axial strains, independent of strain rate and laboratory age.

3. For decreasing strain rates, the bond strength decreased from test 16 to test 17 and then increased from tests 17 to tests 19 and 18. The strength of sample 18 was about[x] equal to that of sample 16. There was no tendency for the soil to become steadily weaker over the full range of strain rates. In fact, the results can be interpreted as indicating a decrease followed by an increase of strength, the increase possibly resulting from the reformation of broken cementation bonds. This is speculation and much more work is required.

---

x) In error the cell pressure for test 18 was 4% larger than the other tests, due to a 10 cm level difference between the table on which sample 18 was being loaded and the compression apparatus in which the other samples were tested. In Fig. 5-3 $(\sigma_a - \sigma_r)$ is plotted correctly for all tests, but for test 18 (σ_a' / σ_r') is overestimated by 4%.

In Fig. 5-4 data are presented from test 18 concerning time and deformation during each stage of creep, together with the results of similar tests on two other samples (Table 5-1), test F.306-8 having been performed on a natural, insensitive, possibly weakly-bonded clay and test F.306-18 having been performed on the same soil first consolidated at 6 kg/cm^2 to destroy all cementation bonds and then allowed to swell at a cell pressure of 0.31 kg/cm^2. In spite of the great differences between these three samples, they all behaved somewhat similarly, strain continuing with time at all stress levels during the periods of observation.

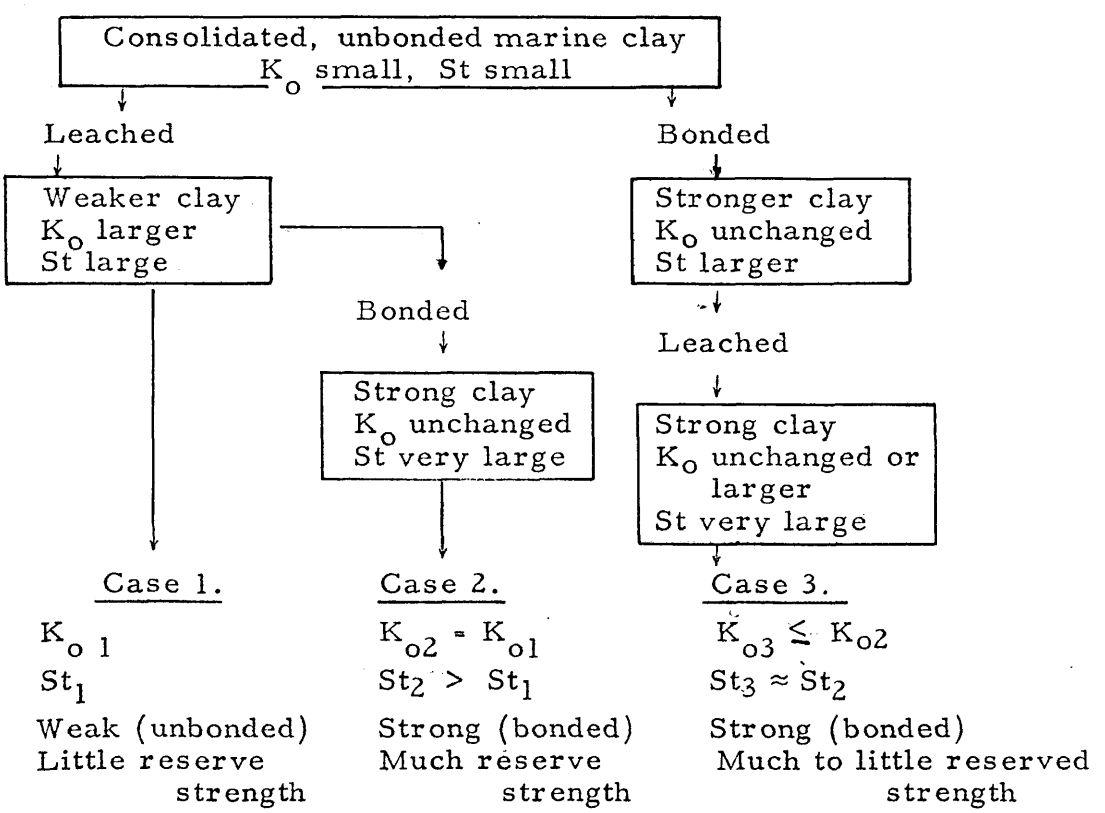
From all the test results concerning the influence of strain rate on strength measured with respect to effective stresses, it is concluded that strain rate has very little influence on the frictional resistance beyond the very rapid rates but it appears to have an effect on the cementation-bond strength, perhaps reflecting the rates of bond breakdown due to shear and the rate of bond reformation. Therefore, variation of drained strength with time should only be dependent on the manner in which bond strength is dependent on time or rate of strain, and any changes of undrained strength with time should be due to changes of pore-water pressure (Bjerrum, Simons and Torblaa, 1958), and to changes of bond strength.

There have been published the results of drained field vane tests in quick clay (Aas, 1966) that could be interpreted as indicating a decrease of bond strength with decreasing rate of strain. However, this type of test is in reality a constant-volume drained test in which volume decreases in the most highly stressed clay are balanced by swelling of the adjacent soil, this causing arching of the soil surrounding the vane and a decrease of normal stress on the potential failure surface. Hence, these decreases of drained strength could be a reflection of decreases of normal stress and might not be a result of changes of the strength properties of the soils.

3. Measurement of K_o.

K_o is a measure of the radial horizontal stress resulting from the existence of a vertical stress for conditions of zero radial strain. Hence $K_o \cdot \sigma_{vert}'$ is a reaction stress and is dependent on the magnitude of the mobilized shear resistance in the soil. If, for some reason, the mobilized shear resistance decreased, the result would be an increase of K_o. Increases of mobilized shear resistance for conditions of constant vertical stress and zero radial strain cannot occur (except under very special conditions, such as internal volume decreases due to syneresis), and it follows, therefore, that under constant vertical stress K_o is the result of the minimum shear resistance mobilized during the history of the soil.

How would cementation or weld bonds affect the magnitude of K_o in leached marine clays? Consider the following possibilities for development of a leached marine clay.



Thus a clay that has never been bonded or possesses very weak bonds (Case 1), and this probably covers most Norwegian quick

clays, might be expected to have the largest values of K_o. A similar clay that became bonded after leaching, and this might have been the case for Labrador clay, would be more sensitive than Case 1 because of a higher undisturbed strength and would have the same K_o value. Case 3 is a clay that was bonded before leaching, and depending on the strength of the bonds, K_o could remain unchanged from the value existing prior to leaching, or if the bonds were broken during leaching (due to settlement strains or dissolution), K_o could become equal to K_o for Cases 1 and 2. In this discussion, that has been limited to normally-consolidated soils, it is shown that it is possible that similar values of K_o can be measured in quick clays having dissimilar properties (Cases 1 and 2), and that dissimilar K_o values can be measured in quick clays having similar properties (Cases 2 and 3), depending upon property changes and the time of their occurrence during the clays' histories.

In field measurements of K_o, soil disturbance would cause arching and an underestimate of K_o. These errors would be most serious in Case 2 where the soil has the largest reserve of strength and is most capable of arching, possibly would be serious in Case 3, and they would be least serious in Case 1 (Norwegian soils). From the experience gained with K_o measurements in Norwegian soils (Chapter 2), and in particular at the Manglerud site where considerable disturbance occurred, it can be anticipated that small variations of degrees of disturbance of the soil along the measurement apparatus could easily be of greater influence than the differences of strength of the soils. However, all else being equal, the errors in K_o measurements in stiff quick clays should be largest.

Natural quick-clay samples in laboratory K_o tests will act as bonded clays to different degrees, and the natural bonds will be more influential in resisting the applied shear stresses than they were in situ, due primarily to the fact that bonds in nature are formed during or after stress application, whereas in the laboratory sample they are in existence before the sample is stressed. Also, due to the difference in time, bonds that are operative in transferring stress in the laboratory might relax due to recrystallization in the field. Thus laboratory tests can be expected to give underestimates of K_o for all types of bonded soil, and this error is not a result of disturbance of the soil.

Unfortunately, laboratory K_o tests have not been made on soils from either Tönsberg or Kjelsås, the two sites at which K_o was successfully measured in situ, and therefore the magnitude of the error is not known.

4. Triaxial Compression and Direct Simple-Shear Tests.

In the introductory chapter of this thesis it was mentioned that the results of K_o-consolidation triaxial compression tests on quick clays always yielded values of undrained shear strength that were larger than those measured in direct simple-shear, vane, unconfined compression or cone tests. This result has been generally accepted as being the result of sample disturbance resulting from reconsolidation. In fact this is not the reason for the difference of the test results, and the reason lies in the interpretation of what is actually being measured.

Consider first the triaxial compression test results on samples of cemented Labrador clay, given in Fig. 5-5. Diagram A presents the results of some drained tests and diagram B the results of some undrained tests. Information concerning the characteristics of the samples is given in Table 5-2. The samples were consolidated isotropically at pressures varying from appreciably smaller to larger than the equivalent isotropic preconsolidation pressure (see compressibility characteristics in Fig. 5-3). At the small consolidation pressures the cementation bonds were not damaged but for increasing values of consolidation pressure greater volume decreases occurred and more cementation bonds were destroyed. Thus, for the larger consolidation pressures the samples behaved more as frictional materials than at the small consolidation pressures.

In the standard triaxial compression test one measures directly the principal stresses, σ_a' and σ_r', and the maximum shear stress in the sample, $\frac{1}{2}(\sigma_a' - \sigma_r')$. The shear resistance of a soil mobilized at any strain can be expressed as :

$$s_m = b_m + \sigma_n' \operatorname{tg} \varphi_m \quad \ldots\ldots\ldots\ldots \quad 5.1$$

in which s_m is the mobilized shear resistance, b_m is the mobilized

cementation-bond resistance, σ_n' is the normal effective stress on the plane being considered, and tg φ_m is the mobilized friction coefficient. From this equation an expression can be derived for the maximum shear stress in a sample in terms of the minor principal effective stress σ_r' and the mobilized shear strength parameters.

$$\left(\frac{\sigma_a - \sigma_r}{2}\right) = b_m\left(\frac{\cos\varphi_m}{1-\sin\varphi_m}\right) + \sigma_r'\left(\frac{\sin\varphi_m}{1-\sin\varphi_m}\right) \quad \ldots . \qquad 5.2$$

In the drained tests shown in Fig. 5-5, σ_r' was constant throughout the test, and for this condition initial failure (that is, initial s_{max}) occurs at a strain when $\frac{d}{d\epsilon}\left(\frac{\sigma_a-\sigma_r}{2}\right) = 0$, and from equation 5.2 the following expression is obtained.

$$\text{At failure,} \quad \left(\frac{\cos\varphi_m}{1-\sin\varphi_m}\right)\frac{d}{d\epsilon}[b_m] + b_m\frac{d}{d\epsilon}\left[\frac{\cos\varphi_m}{1-\sin\varphi_m}\right]$$

$$+ \sigma_r'\frac{d}{d\epsilon}\left[\frac{\sin\varphi_m}{1-\sin\varphi_m}\right] = 0 \quad \ldots . \qquad 5.3$$

This expression indicates that an unbonded soil, in which $b_m = 0$, will fail when friction is fully mobilized, and test 11 in Fig. 5-5 is an example of this behaviour, the natural cementation bonds having been destroyed during consolidation. For a bonded soil failure first occurs when the bond strength is decreasing at the same rate as the mobilized friction is increasing, and the strength of the soil will decrease if the rate of decrease of bond strength is greater than the increase of frictional strength, as indicated in tests 16 and 7 in Fig. 5-5. The strength of the soil could increase again after an initial failure if, for increasing strain, the rate of decrease of bond strength became less than the rate of increase of frictional strength, as indicated by test 12 and by the triaxial test given in Fig. 5-6.A.

In tests in which the radial effective stress σ_r' is varied or varies because of changing pore-water pressures (undrained tests), the maximum strength of a sample in the triaxial test is dependent on the same factors that governed the soil behaviour in standard drained tests (mobilization of bonds and friction strengths) and in addition is dependent on changes of effective stress. This behaviour can be expressed by setting $\frac{d}{d\epsilon}\left(\frac{\sigma_a-\sigma_r}{2}\right) = 0$, obtainable from equation 5.2. At failure,

$$\left(\frac{\cos\varphi_m}{1-\sin\varphi}\right)\frac{d}{d\epsilon}[b_m] + b_m\frac{d}{d\epsilon}\left[\frac{\cos\varphi_m}{1-\sin\varphi_m}\right]$$

$$\left(\frac{\sin\varphi_m}{1-\sin\varphi_m}\right)\frac{d}{d\epsilon}[\sigma_r'] + \sigma_r'\frac{d}{d\epsilon}\left[\frac{\sin\varphi_m}{1-\sin\varphi_m}\right] = 0 \quad . \quad . \quad . \qquad 5.4$$

If the soil is unbonded ($b_m = 0$) and therefore exhibits a frictional behaviour the soil will have maximum strength when the rate of decrease of effective stress is equal to the rate of mobilization of friction, and the strength will decrease for increasing strain if the rate of decrease of effective stress is larger than the rate of increase of mobilized friction. The effect of a bond strength component would be to increase the strength at failure and probably increase the rate of strength decrease after failure. Examples of these behaviours are given in Fig. 5-5.B (all samples have bond strength) and in Fig. 5-7.A where the sample tested at $P_c'/P_o' \approx 3$ can be considered as having no cementation bonds.

In contrast to triaxial testing, in the direct simple-shear test (Bjerrum and Landva, 1966) direct measurements of principal stresses cannot be made. During consolidation for conditions of zero applied horizontal shear stress the vertical and horizontal stresses acting on the sample are principal stresses and are equal to those in triaxial tests for the condition of K_o consolidation, as directly measured with the use of the reinforced rubber membranes described in Chapter 2. Results of K_o consolidation prior to triaxial and direct simple-shear tests are presented in Figs. 5-6 and 5-7 and they show that for similar axial stress, the respective principal stresses in both triaxial and direct-shear tests are equal. In the direct simple-shear test the sample is brought to failure by applying horizontal shear stresses. This results in rotation of the principal stresses, and at one point during the test when they have rotated 45^o the applied shear stress on horizontal planes will be equal to the maximum shear stress in the sample. Both before that point in the test, and after, the applied shear stress on horizontal planes will be less than the maximum shear stress existing in the sample, and for this reason it is clear that stress measurements made on horizontal planes in direct simple-shear tests cannot be directly compared with measured maximum

shear stresses in triaxial tests.

Professor J.H. Schmertmann has developed a very refined stress-analysis method for estimating the principal stresses during an NGI direct simple-shear test, using measured values of applied vertical stress, applied shear stress, average radial stress in the confining membrane, and horizontal strain (Schmertmann, 1962, 1963). The method is not exact but the errors involved are probably of similar magnitude to the inaccuracies of the measurement apparatus. With the use of this method, principal stresses have been estimated for two of the tests that are given in Figs. 5-6.C and 5-7.B. These results indicate clearly the dissimilarity between the shear stress on horizontal planes (τ_h) and the maximum shear stress within the sample $\frac{1}{2}(\sigma_1-\sigma_3)$, and they also show the similarity between the results of direct simple-shear tests and triaxial compression tests when compared in terms of principal stresses. Of most importance, these results indicated that in the direct-shear tests initial failure occurred very early during the tests on planes other than horizontal when the shear stress on horizontal planes was still small and increasing, and this finding will be considered more fully later.

The shape of the τ_{max} curve in a direct-shear test is dependent on the same factors that govern the τ_{max} curve in triaxial tests; namely changes of σ_3' and changes of the magnitudes of mobilized bond strength and mobilized friction, as previously discussed with respect to triaxial tests. The shape of the τ_h curve (shear stress on horizontal planes) in direct-shear tests is similarly dependent but there are several additional contributing factors. One of these is that the horizontal planes in direct-shear tests are not the planes on which failure first occurs, and if there is a cementation-bond strength component and this component is isotropic in nature, it follows that it will begin to decrease (to be destroyed) at small strains when failure first begins to occur in the sample and that its magnitude will have been reduced by the time the horizontal plane becomes the most critically-stressed plane. The other factor is that the pore-water pressure acting in undrained tests on horizontal planes is the result of critical shear stresses having acted on other planes. This is clearly shown in Fig. 5-7.B; at small shear strains ($\approx 1\%$) when failure first

occurred the pore-water pressures were very small, but when the principal stresses had rotated such that the horizontal planes became the most highly shear-stressed planes (strain ≈ 10%) the pore-water pressures had increased significantly. Therefore, when considering the behaviour of soil in this type of simple-shear test, caution must be taken in interpreting the results; the measurements of stresses on horizontal planes give a measure of behaviour, and this behaviour is dependent on the properties of the soil but also on magnitudes and directions of principal stresses and changes of soil properties during the test.

5. The Furre Slide.

The soil conditions at Furre were rather special (Hutchinson, 1961). The terrace that bordered the Namsen River consisted primarily of sand but contained several thin (less than 10 cm thick) continuous layers of quick clay that dipped down towards the river at an average slope of 6 degrees, being 3 degrees below the river and 9 degrees at the back edge of the slide. As the result of a small slide in the river bank, a large part of the terrace, measuring about 200 by 800 metres, slid towards the river along one of the clay layers. It is not difficult to understand that this slide led to the development of the NGI direct simple-shear equipment.

A drained field shear-box test was performed on the clay layer that was exposed at the rear of the slide area, where the clay layer was 10 cm thick and was inclined at 8 degrees (Hutchinson, 1961; Hutchinson and Rolfsen, 1962). The test apparatus consisted of an open box divided into an upper and a lower half. The box was lowered so that the junction of the two halves was located within the clay layer and the clay was loaded vertically to approximately the pre-slide overburden pressure; that is, $P_c' = P_o'$. Shear forces were applied to the upper half of the box in a direction parallel to the plane between the two halves of the box, and the shear forces were maintained constant during each load increment by means of a cable-and-spring arrangement. The shear load was built up by using many small load increments and for shear stresses exceeding

$\tau_h/P_c' = 0.14$ large vertical and horizontal[x] deformations occurred (see test result, Fig. 5-6.D). When the test was terminated there was a strong indication that the shear resistance of the soil would increase for further strain.

When this result is compared to the direct simple-shear test results in Fig. 5-6, it is realized that the measured τ_h must have been smaller than the shear stresses on other planes within the soil. If we assume, for example, that K_o for the Furre clay was equal to that at Kjelsås, the maximum shear stress in the soil for K_o-conditions would have been $\tau/P' = \frac{1}{2}(1-K_o) \approx 0.17$, and for failure to occur within the soil in a drained or undrained state the shear stress would have had to be larger than this value. Another way to consider the Furre test is to say that the mobilized shear resistance on the horizontal plane is equal to the mobilized bond strength plus the mobilized frictional resistance:

$$\tau_h = b_m + \sigma_n' \operatorname{tg} \varphi_m' \quad \ldots \ldots \ldots \ldots \ldots \quad 5.5$$

Between horizontal deformations of 1 and 4 mm τ_h was about constant and therefore $\frac{d}{d\epsilon}(\tau_h) = 0$, that is:

$$\frac{d}{d\epsilon}(b_m) + \sigma_n' \frac{d}{d\epsilon}(\operatorname{tg} \varphi_m') = 0 \quad \ldots \ldots \ldots \quad 5.6$$

Because it is known that at such a small value as $\tau_h/P' = 0.16$, $\operatorname{tg} \varphi_m'$ is still increasing, the test result indicates that for the horizontal strain interval from 1 to 4 mm the bond strength was decreasing at a rate equal to the increase of mobilized friction. If bond strength was decreasing on the horizontal plane, it must have been larger at smaller strains on planes on which failure was occurring. Some indication of this can be gained by considering that the volume changes that occurred during the test are to a large extent a reflection of the breakdown of brittle bonds between particles, and that the mobilized shear resistance on the horizontal planes for small horizontal deformations is dependent largely on these same brittle bonds. The lower diagram in Fig. 5-6.D indicates a marked weakening of the

x) The plane on which shear stresses were applied was not horizontal but was inclined at 8^o. The direction of the applied shear stresses was uphill.

structure (settlement) at $\tau_h/P_c' \approx 0.14$ (interpreted as being due to bond-strength reduction) and that during the phase of the test when large settlements occurred there was very little increase of τ_h/P_c' (interpreted as showing the influence of the decrease of bond strength on the shear resistance that could be mobilized, as expressed in equation 5.6).

The behaviour of the field direct-shear test gives a very clear picture of the state of the clay before the slide occurred and of its behaviour during the slide. Due to its natural inclination, the clay was subjected to an average shear stress parallel to the plane of sliding equal to 0.12 of the overburden effective stresses, and in this state it was in a very delicate state of equilibrium, as evidenced by the behaviour of the field direct-shear test at this stress condition. Being normally consolidated and possessing very large sensitivities, the clay had very little reserve strength, and increases of shear stress would cause shear strains, destruction of bonds, increases of compressibility, increases of pore-water pressure, and decrease of volume if water was able to drain away. This was the clay's behaviour during the field test, but if during the test the shear-stress increments had been increased, and possibly only slightly, there is little doubt that the test sample would have failed in undrained shear due to the rapid build-up of water pressures and their inability to dissipate sufficiently quickly. It is believed that the Furre slide occurred by just this mechanism of a sudden change from drained to undrained shear conditions. As the result of the initial slide that occurred in the river bank, the thin layer of quick clay around the initial-slide boundary became locally overstressed, and because the reserve strength of the soil was less than the local increase of shear stresses, it failed in undrained shear. Upon failure its strength decreased, causing the transference of shear stress to clay further along the layer, and in this manner failure could have spread. Because the clay layer was very thin, local horizontal deformations of several millimeters of the overlying sand body relative to the sand underlaying the clay were all that was required to allow the failure front in the clay to progress. It could be speculated that had the entire soil profile consisted of soft quick clay, a retrogressive slide of the Selnes or Göta types would have occurred.

In the author's opinion the significant factors that dictated the behaviour of the soils involved in the Furre slide were (i) the small reserve strength for undrained conditions of normally-consolidated quick clays, such as Furre, (ii) the strain-sensitive nature of the soil that causes large strength reductions following undrained failure, (iii) the confinement of a thin layer of quick clay between large masses of strain-insensitive sand that caused the shear strains to be concentrated within the most strain-sensitive material, the clay, and (iv) the occurrence of an initial slide that caused sudden local applications of shear stress.

6. Anisotropy of Undrained Shear Strength.

Frequently in design, use is made of undrained shear strength to estimate soil stability. Usually it is assumed that in the undrained state soil will behave as an isotropic cohesive material and therefore a single value of strength, measured by means of undrained compression, direct-shear or vane tests, is sufficient to describe the strength behaviour of the soil. For most types of problems involving undrained (rapid) loading, this approach has proved to be successful and practical. However, it is now known that undrained strength is not necessarily isotropic and it is known that all types of undrained tests do not necessarily give the same strength result, and therefore the use of undrained strength in stability analyses is not as straightforward as previously thought.

Aas (1965, 1966) has directly measured anisotropy of undrained strength in the field by using vanes of different shapes and calculating the resistance developed on the vertical and horizontal failure surfaces. The results indicate that the degree of strength anisotropy is related in magnitude to the ratio of the natural in situ effective stresses. For standard shaped vanes the average strength measured is only slightly greater than that developed on vertical planes, and it can be appreciably smaller than the strength that can be developed on horizontal planes.

A comparison of the results of K_o-consolidated undrained triaxial compression tests and of constant-volume direct simple-shear tests

indicates that differences of undrained strength can be expected in planes of different inclination in the ground. In Fig. 5-7 the triaxial test at $P_c'/P_o' \approx 3$ gave $\tau_{max}/P_c \approx 0.28$ for a plane inclined at 45^o, and about the same result was obtained for 45^o planes from the direct-shear test in the same figure, whereas on horizontal planes in the direct-shear test a value of only $\tau_{max}/P_c' \approx 0.23$ was obtained.

In addition, there is the possibility that two identical soil elements on planes of identical inclination and subjected to identical consolidation stresses will exhibit different behaviours if sheared in different directions. This possibility is shown in Fig. 5-8 where two elements on a slip surface are failed by the application of a load. Direct simple-shear tests were performed to study the behaviours of such elements and the results are given in Fig. 5-8 and the soil information is given in Table 5-3. The samples were consolidated under identical vertical and horizontal shear stresses, and then they were brought to failure in constant-volume tests, one sample being deformed in the same direction as the consolidation shear stress (element A) and the other sample being deformed in the opposite direction (element B). The samples consolidated under vertical pressures equal to P_o' (bonds not destroyed) gave strengths equal to $s_u/P_c' = 0.33$ (element A) and 0.24 (element B), and the samples consolidated at $P_c' \approx 2\ P_o'$ (some bonds destroyed) gave strengths equal to $s_u/P_c' = 0.27$ (element A) and 0.15 (element B). These strength differences are the results of the differences of strain required to cause failure. In those tests in which failure was caused by reversing the direction of applied shear stress, additional strain was required because the strength parameters were first demobilized and then remobilized, and the additional strain caused larger pore-water pressures, smaller effective stresses, increased destruction of cementation bonds, and therefore smaller undrained shear strengths. The influence of the direction of stress changes during undrained shear has also been investigated by Ladd and Bailey (1964) and their finding was also that this factor could be of considerable importance.

Very little work has been done to determine the degree of strength anisotropy in situ, and therefore it is unknown whether the general problem is serious or not. It can be anticipated that the severity of the problem is least in slightly-overconsolidated soils in which the degree of stress anisotropy is small or in highly-bonded soils, and it can be anticipated to be greatest in soils in which the degree of stress anisotropy is large (normally-consolidated and heavily-overconsolidated soils). To study the seriousness of the problem in Norwegian soils, special field apparatus has been constructed by the Norwegian Geotechnical Institute to perform in situ direct-shear tests at different depths on horizontal planes and in two directions (downward and upward) on 45° planes. To date (1966) only tests have been performed on horizontal planes.

7. Strength Determination by Vane Tests.

As mentioned previously, field vane tests in which vanes of different shapes were used showed that the strengths mobilized on vertical and horizontal planes could be different in magnitude, and this difference appeared to be related to the ratio of in situ principal effective stresses (Aas, 1965, 1966). This finding was followed up by a program of laboratory vane-triaxial tests in which undisturbed samples of clay could be subjected to different consolidation stresses and vane tests could be performed with vanes of different shapes to determine the degree of strength anisotropy. The vane-triaxial apparatus was developed by Landva (1962) and an improved model that was used in this program has been described by Kenney and Landva (1965). The vanes used varied in diameter and height from D/H = 8/32 mm to 16/8 mm. The rate of vane rotation used in all the tests was 6°/minute. The soils were undisturbed samples of Manglerud III and Tönsberg clays, and the consolidation stresses were appreciably larger than the in situ stresses in order that any natural bond strength of the soil would be destroyed; it was feared that the use of different consolidation stresses would cause different degrees of bond destruction if these bonds were allowed to exist, and therefore to remove this possibility all bonds were destroyed.

In the first series of tests samples were consolidated at different values of $K_c = \sigma_{rc}'/\sigma_{ac}'$ and vane tests were made with a standard-shaped vane having D/H = 0.5. The shear strength was initially estimated on the basis of the usual assumption made in practice that the shear strengths that could be developed on vertical and horizontal planes, s_v and s_h respectively, were equal. This average undrained shear strength, $\bar{s}_u$, was estimated from the expression:

$$\frac{\bar{s}_u}{\sigma_{ac}'} = \frac{2M}{\pi D^2 H} \frac{1}{\sigma_{ac}'} /\left(1 + \alpha \frac{D}{H}\right) \quad \ldots\ldots\ldots 5.7$$

in which M is the applied moment and α is a shear-stress distribution factor for the end failure surfaces that was assumed to be equal to o.3. The results are presented in Fig. 5-9.A for tests performed immediately after the vane was inserted (undrained tests) and for tests in which reconsolidation of the soil was allowed after vane insertion and before the test was performed (reconsolidation tests). The results in the upper two diagrams clearly indicated that the strength ratio $\bar{s}_u/\sigma_{ac}'$ was dependent on the ratio of consolidation stresses and that for larger values of K (larger average stress) there was measured larger values of $\bar{s}_u/\sigma_{ac}'$. Another result was that if reconsolidation was allowed to occur before the test, significant increases of strength could be obtained, and this result had been previously reported from field tests by Flaate (1966).

In the lower two diagrams the strength results are plotted against the maximum shear stress existing within the sample during consolidation, where $\tau_{max}/\sigma_{ac}' = \frac{1}{2}(1 - K_c)$. For the condition during consolidation when there were zero shear stresses within the soil ($K_c = 1.0$), the value of $\bar{s}_u/\sigma_{ac}'$ was a maximum, whereas for increased shear stresses during consolidation (smaller values of K_c) the strength ratio decreased. In the case of Manglerud III clay, for values of K_c smaller than 0.55, it is shown that $\bar{s}_u/\sigma_{ac}' < \tau_{max}/\sigma_{ac}'$; that is, the determined undrained shear strength of the soil was less than the maximum shear stress in the soil, meaning that use of the undrained shear strength could not predict the obvious stability of samples consolidated at K_c values less than 0.55.

Because K_c in natural slopes is probably appreciably less than 0.5, it is not surprising that stability analyses of stable natural slopes based on vane strengths have sometimes indicated safety factors less than unity (Bjerrum and Kjærnsli, 1957).

The second series of tests was performed on samples consolidated at different ratios of consolidation stresses from $K_c = 1.0$ to $K_c = 0.4$ using vanes of different shapes in order to determine the degree of strength anisotropy. The tests were interpreted from the following equation:

$$M = \frac{\pi}{2} HD^2 s_v + \frac{\pi}{2} D^3 \alpha s_h$$

and this can be written as:

$$\frac{2M}{\pi D^2 H} \frac{1}{\sigma_{ac}'} = \frac{s_v}{\sigma_{ac}'} + \alpha \frac{s_h}{\sigma_{ac}'} \frac{D}{H} \quad \ldots\ldots\ldots 5.8$$

This equation plots as an inclined straight line, as shown in the upper diagram of Fig. 5-9.B, the slope of which is equal to $\alpha \frac{s_h}{\sigma_{ac}'}$ and the intercept is equal to $\frac{s_v}{\sigma_{ac}'}$. The test results indicated that the ratio of the shear strength on horizontal planes to the axial stress acting normal to these planes (s_h/σ_{ac}') was independent of the radial stress acting on the sample (independent of K_c), and that the shear strength on vertical planes was dependent on K_c.

With the use of equation 5.8, and by using the finding from the test series given in Fig. 5-9.B that s_h/σ_{ac}' for $K_c = 1.0$ is equal to $s_v/\sigma_{ac}' = \bar{s}_u/\sigma_{ac}'$, the strength ratio s_v/σ_{ac}' was calculated from the undrained test results given in Fig. 5-9.A, and these values are plotted in the upper diagram of Fig. 5-9.C. They indicate that the shear strength on vertical planes is strongly dependent on the normal stress acting on these planes during consolidation. In the case of Manglerud III the relationship is one of direct proportion. These results have been replotted in the lower diagram as degree of strength anisotropy, s_v/s_h, plotted against consolidation stress anisotropy, $\sigma_{rc}'/\sigma_{ac}' = K_c$. The results clearly show that strength anisotropy is strongly influenced by consolidation-stress

anisotropy, and supports the intuitive conclusions on this matter made by Aas (1965, 1966).

8. Conclusions.

The theme of this chapter has been the behaviour of natural quick clays, considered with respect to the properties of clays and their stress, strain and drainage environments. Shear resistance has been divided for convenience into two independent components, frictional resistance that is dependent on particle properties and also on effective stress, and cementation bonds, that are pictured as being strain sensitive and dependent perhaps on time. Each of these components is mobilized through shear deformation. but probably in different ways; friction could be imagined as becoming mobilized through the mutual sliding of particles, causing a denser soil fabric, and cementation bonds could be thought of as being destroyed by the same physical process. Thus these two components should be considered separately when soil behaviour is to be evaluated.

Although very little is now (1966) known about the cementation-type bonds in Norwegian quick clays, these bonds seem to contribute a significant proportion of the strength of quick clays at small strains. In contrast to most other soils, the cohesive contribution can decrease rapidly with increasing strain, such that it can cause a decrease of the total strength of the clay in spite of increases of mobilized frictional strength, and this applies to both drained and undrained shear conditions. This means that in design problems concerning quick clays it should be assumed that the clays are bonded, and attention should be directed towards the changes that could occur to the magnitude of these bonds. If the soils are overconsolidated and the bonds are strong, the reserve strength of the clay will be large and the likelihood of conditions changing locally and suddenly from drained to undrained is small. (Chapter 3, eroded slopes). Should the clay be normally consolidated and the bond strength small and strain-sensitive, the reserve strength of the clay will be small and the possibility of local undrained failure occurring, followed by progressive failure, should be considered. In these latter clays, the

strength behaviour will be strongly influenced by conditions of stress and strain in laboratory tests, and therefore it is important that the test conditions simulate as closely as possible the soil conditions in the field.

Table 5-1. Results of rate-of-strain and aging tests on cemented Labrador clay and on Vormsund clay.

Test No.	Date began 1966	Water Content			Strength			Consol-idation		Shear Test					Remarks
		w %	w_L %	w_p %	s_u t/m²	s_u t/m²	St	σ_{rc} kg/cm²	$\frac{\Delta V}{V}$ %	$\sigma_a - \sigma_r$ kg/cm²	σ_a' / σ_r'	Strain ϵ_a %	Strain rate %/day	Age at failure days	
Labrador															
15	7/2	32-37 30.0	31.5	22.9	2.8	0.17	16	0.25	1.5	1.17	5.69	2.3	0.40	85	Aged for 80 days for σ_r = 0.25 kg/cm²
16	7/2	29-33 33.3	28.2	20.5	3.2	0.14	23	0.25	1.5	1.20	5.81	2.4	0.48	12	
17	8/2	32-34 32.6	28.0	20.6	3.5	0.13	27	0.25	1.0	1.04	5.15	2.4	0.088	30	
18	8/2	31-34 33.2	27.8	21.4	3.7	0.10	37	0.26	1.1	1.19	5.58	2.6	-	215	Loaded in stages.
19	9/2	30-34 32.1	30.2	21.8	4.5	0.28	16	0.25	1.0	1.07	5.28	2.3	0.032	77	
Vormsund															
8		38.4 38.6	42.0	26.8	- 2.8	- 0.34	- 8	0.30	1.9	0.79	3.60	5.4	-	153	Loaded in stages
18		36.6 35.5	43.5	30.1	- 3.0	- 0.63	- 5	6.0/0.31	10.6	0.89	3.82	3.3	-	104	Loaded in stages

Note: For all tests, the first line is before-test results, the second line is after-test results.

Table 5-1.

Table 5-2. Summary of results of triaxial compression tests on Labrador clay.

Drained compression tests.

Test No. F.301	Routine tests						Consolidation		Initial failure					Ultimate failure			
	w_L %	w_p %	w %	s_u t/m²	$s_{\bar{u}}$ t/m²	St	σ_{r_c} kg/cm²	$\frac{\Delta V_c}{V_o}$ %	$\sigma_a' - \sigma_r'$ kg/cm²	σ_a'/σ_r'	$\frac{\Delta V_s}{V_c}$ %	ϵ %	Strain rate %/day	$\sigma_a' - \sigma_r'$ kg/cm²	σ_a'/σ_r'	$\frac{\Delta V_s}{V_c}$	ϵ_a %
16	28.2	20.5	29-33				0.25	1.5	1.20	5.81	0.6	2.4	0.5				
			33.3	3.2	0.14	23											
7	28.2	20.6	34-38				0.50	2.6	1.74	4.47	1.0	2.4	1.9				
			31.7	4.6	0.09	50											
9	28.4	20.8	28-34				1.00	2.5	2.25	3.25	1.2	2.9	2.0				
			28.0	6.1	0.57	11											
12	27.0	19.3	33.37				2.00	2.9	3.02	2.51	1.4	2.7	2.7	4.0^+	3.0^+	9.6^+	24^+
			26.4	7.7	0.59	13											
11	27.3	17.1	37-38				4.00	6.0						8.6^+	3.1^+	9.6^+	20^+
			23.4	11.2	2.3	5											

Undrained compression tests.

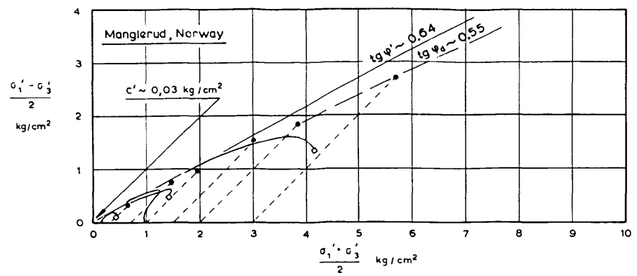
Test No. F.301	Routine tests						Consolidation		Failure $(\sigma_a' - \sigma_r')_{max}$				Failure $(\sigma_a'/\sigma_r')_{max}$				
	w_L %	w_p %	w %	s_u t/m²	$s_{\bar{u}}$ t/m²	St	σ_{re} kg/cm²	$\frac{V_c}{V_o}$ %	$\sigma_a' - \sigma_r'$ kg/cm²	σ_a'/σ_r'	u kg/cm²	ϵ_a %	Strain rate %/hr	$\sigma_a' - \sigma_r'$ kg/cm²	σ_a'/σ_r'	u kg/cm²	ϵ_a %
8	28.5	20.6	35-37				0.50	2.6	1.33	4.47	0.12	2.8	0.12	1.06	5.53	0.27	1.5
			30.8	3.8	0.08	50											
10	26.4	20.7	30-34				1.00	2.8	1.78	4.19	0.44	2.4	0.10	1.60	4.45	0.54	1.5
			30.0	4.3	0.14	30											
14	27.3	19.7	31-33				1.50	2.8	2.09	3.83	0.76	1.9	0.08	2.09	3.83	0.76	1.9
			31.7	4.9	0.11	45											

Note: For all tests, the first line is before-test and test results,
the second line is after-test results.

Table 5-3. Information concerning samples of Lierstranda clay. (see Fig. 5-8).

Test No. F.281-2	Water Content			Strength			Consolidation					Failure	
	w	w_L	w_p	s_u	$s_{\bar{u}}$	St	P_o'	P_c'	τ/P_c'	ϵ_v	ϵ_H	τ_h/P_c'	ϵ_H
	%	%	%	t/m^2	t/m^2		kg/cm^2	kg/cm^2		%	%		%
4	35.8	26.4	19.3	1.00	0.01	100	0.45	0.43	0..25	4.6	2.0	0.33	1.0
	33.8			1.05	0.03	35							
2	37.3	30.4	23.1	1.04	0.01	100	0.44	0.44	-0.25	4.3	1.3	0.24	4.0
	36.1			1.10	0.04	28							
5	37.0	29.0	20.8	1.00	0.01	100	0.45	1.00	0.22	9.1	6.6	0.27	1.0
	33.2			1.80	0.07	26							
6	36.8	26.2	19.7	0.96	0.02	48	0.45	1.00	-0.22	8.8	5.6	0.15	6.0
	28.5			2.27	0.06	38							

Note: For all tests, the first line is before-test results, the second line is after-test results.



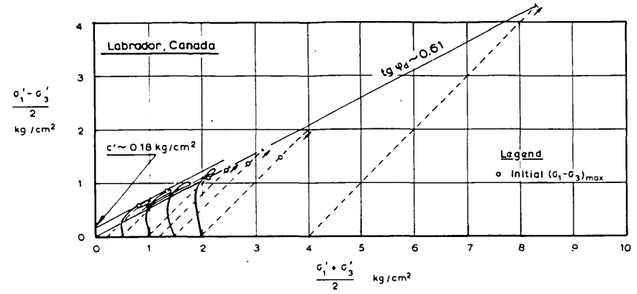
Manglerud, Norway

w = 45 % w_L = 29 % w_P = 20 %

I_L = 2 to 3 < 2μ = 44 %

S_t = 50 to 100

S_u = 1.2 t/m² P_c = 5 t/m²



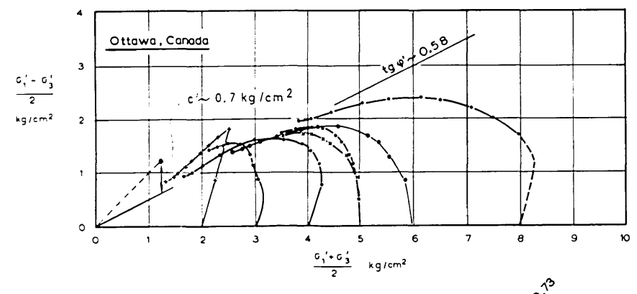
Labrador, Canada

w = 33 % w_L = 27 % w_P = 19 %

I_L = 1.5 to 2 < 2μ = 55 %

S_t = 100

S_u = 7 t/m² P_c = 50 t/m²



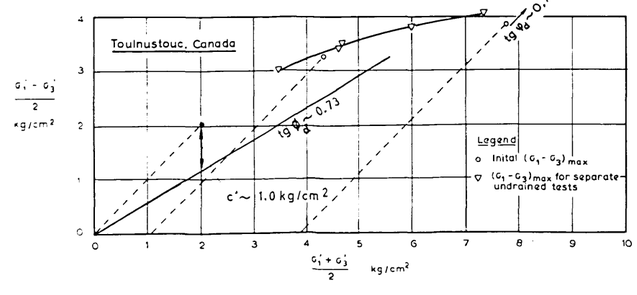
Ottawa, Canada (Crawford, 1963)

w = 53 % w_L = 31 % w_P = 23 %

I_L = 3,5 to 4 < 2μ = 65 %

S_t > 100

S_u = 21 t/m² P_c = 45 t/m²



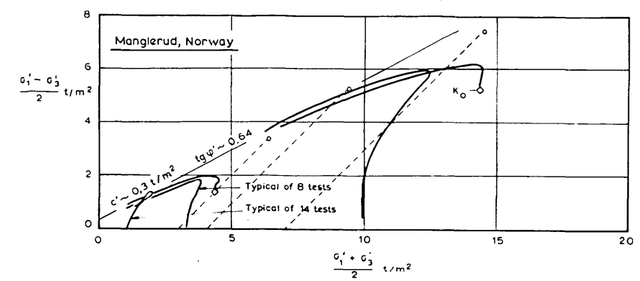
Toulnustouc, Canada (Conlon, 1966)

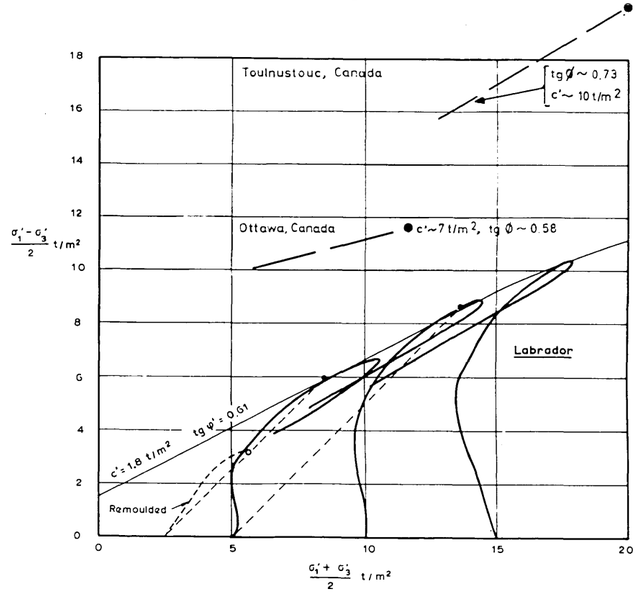
w = 29 % w_L = 21 % w_P = 17 %

I_L = 3 to 4 < 2μ = 25 %

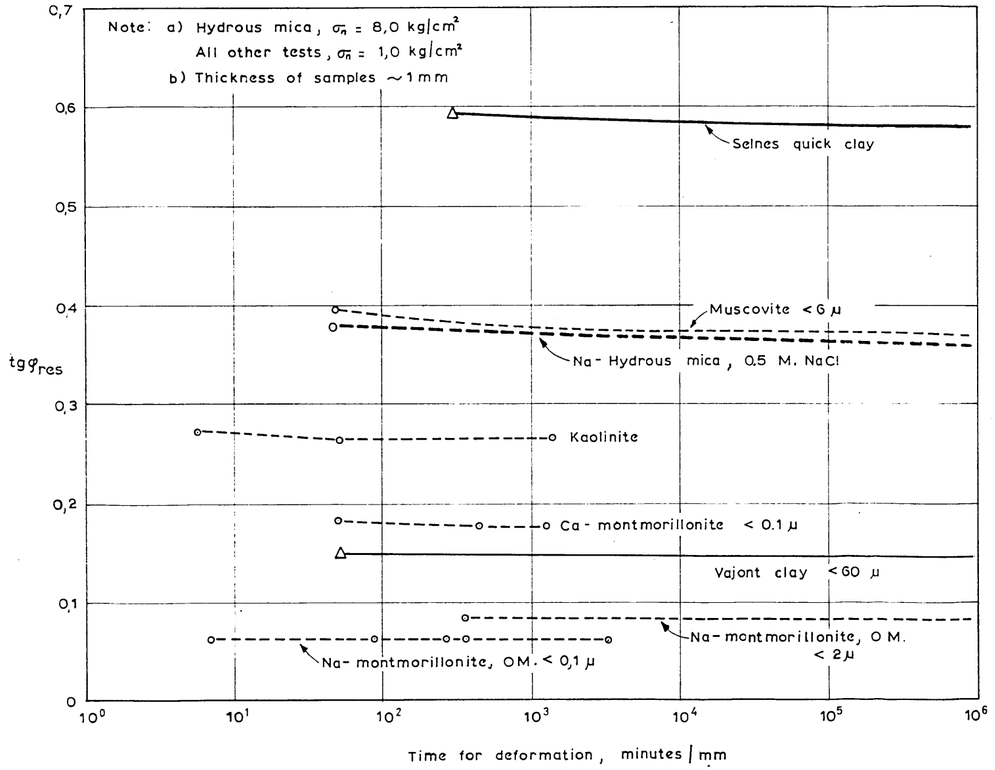
S_t = > 100

S_u = 35 t/m² P_c = 120 t/m²

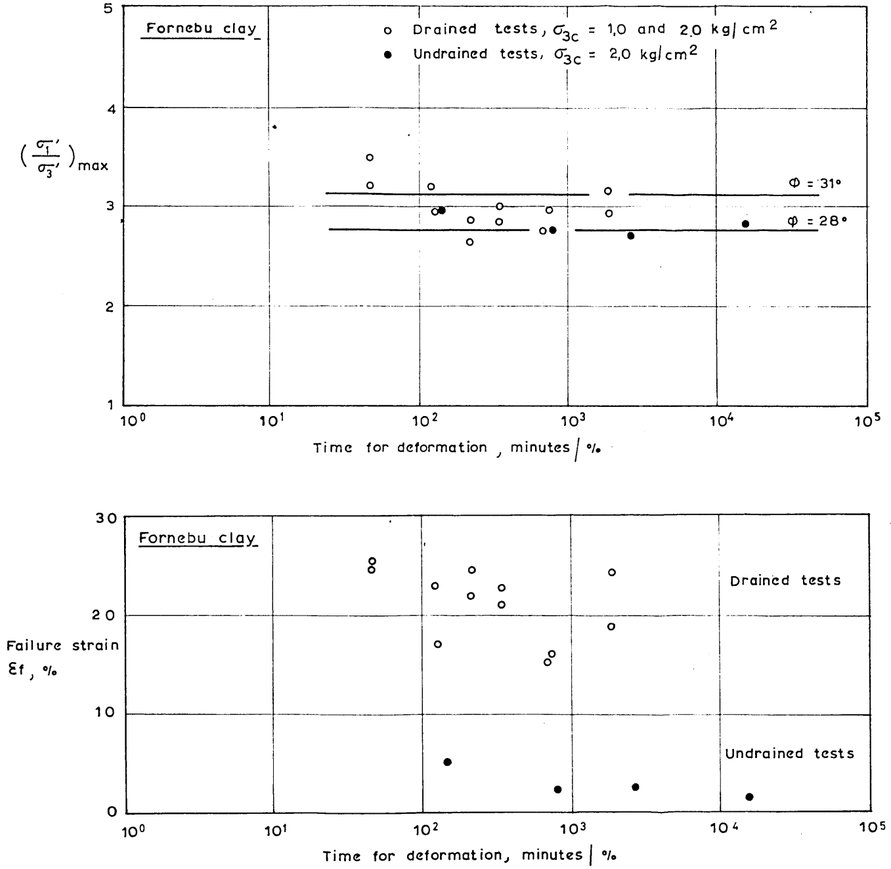




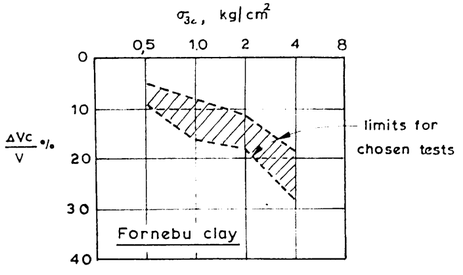
SHEAR STRENGTH BEHAVIOURS OF UNDISTURBED QUICK CLAYS HAVING DIFFERENT DEGREES OF PARTICLE BONDING.



A MINERALS AND SOILS IN THEIR RESIDUAL-STRENGTH STATE

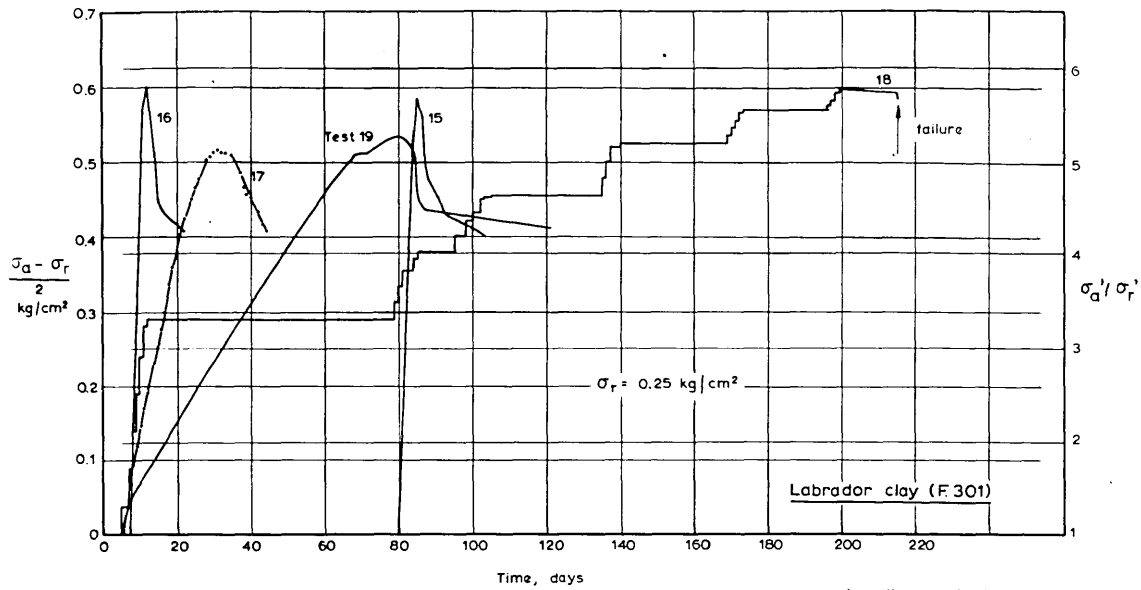


B. NATURAL CLAY CONSOLIDATED AT $\sigma_{3c} \geq P_o$

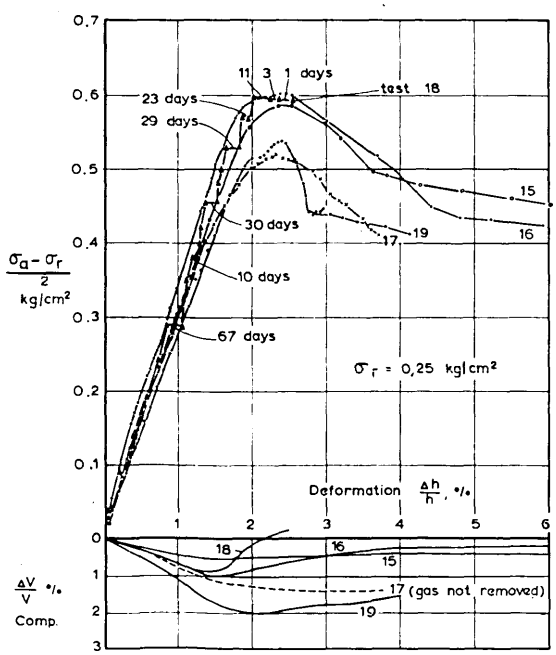


VOLUME REDUCTIONS BEFORE SHEAR TESTS

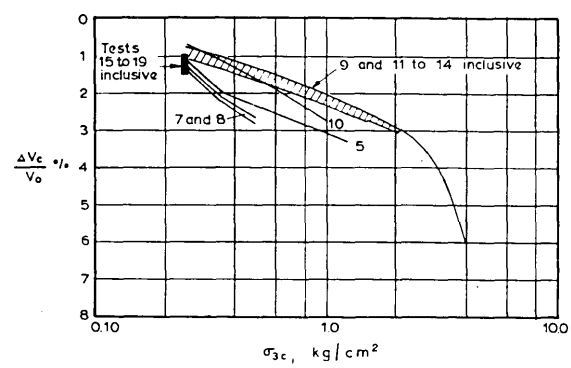
EFFECT OF STRAIN RATE AND TIME ON THE DRAINED SHEAR BEHAVIOUR OF UNCEMENTED CLAYS



Loading schedules

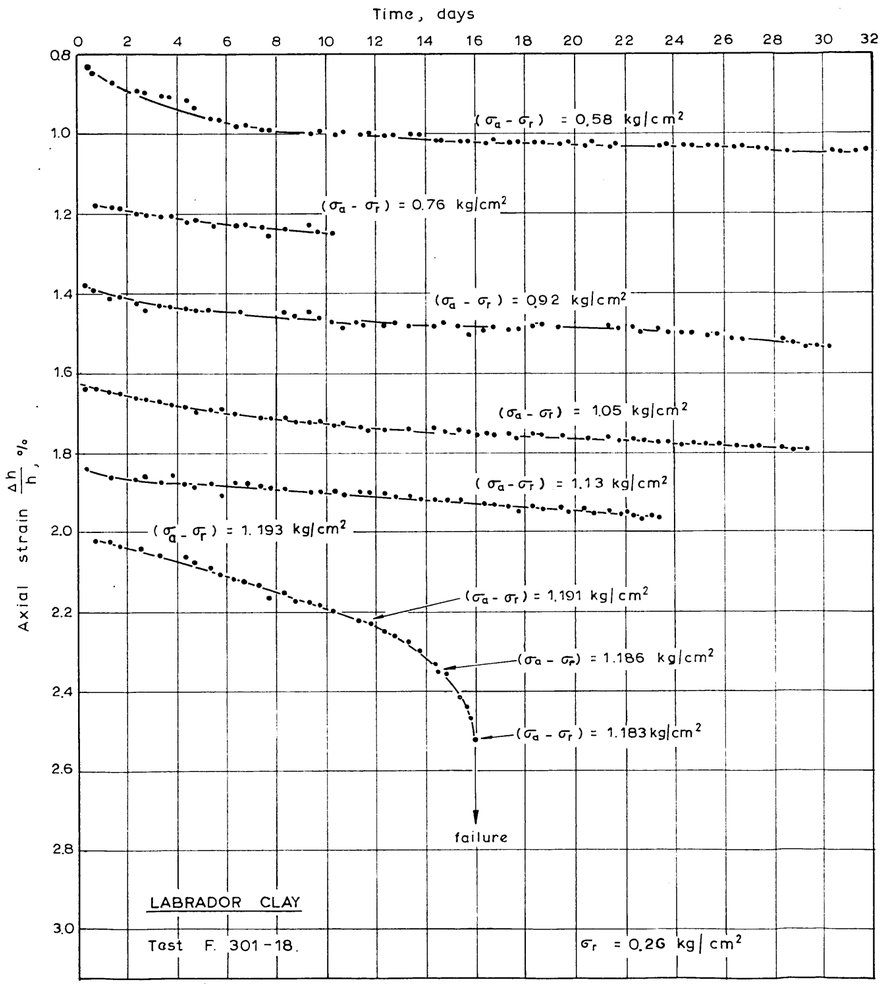


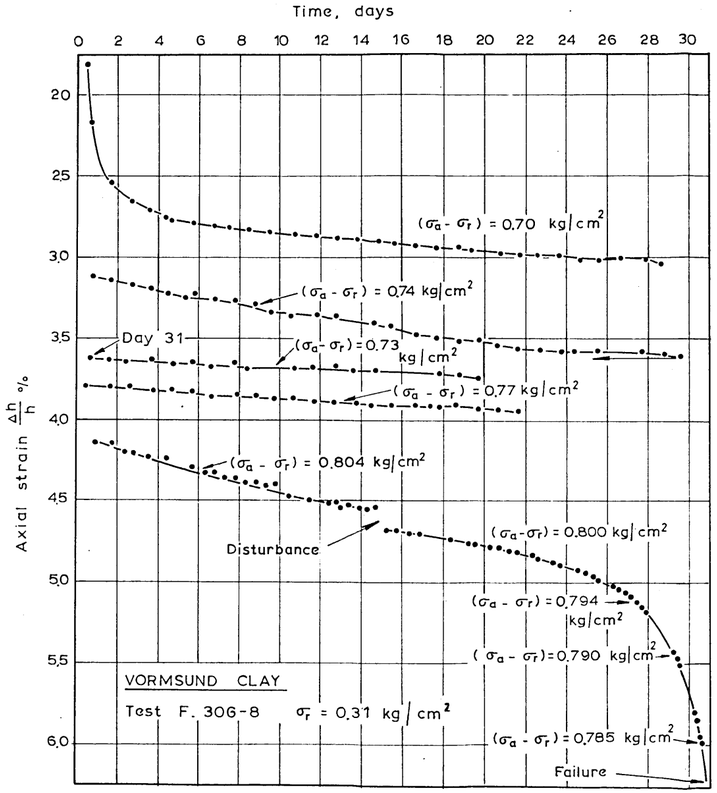
Stress-strain and volume-change results

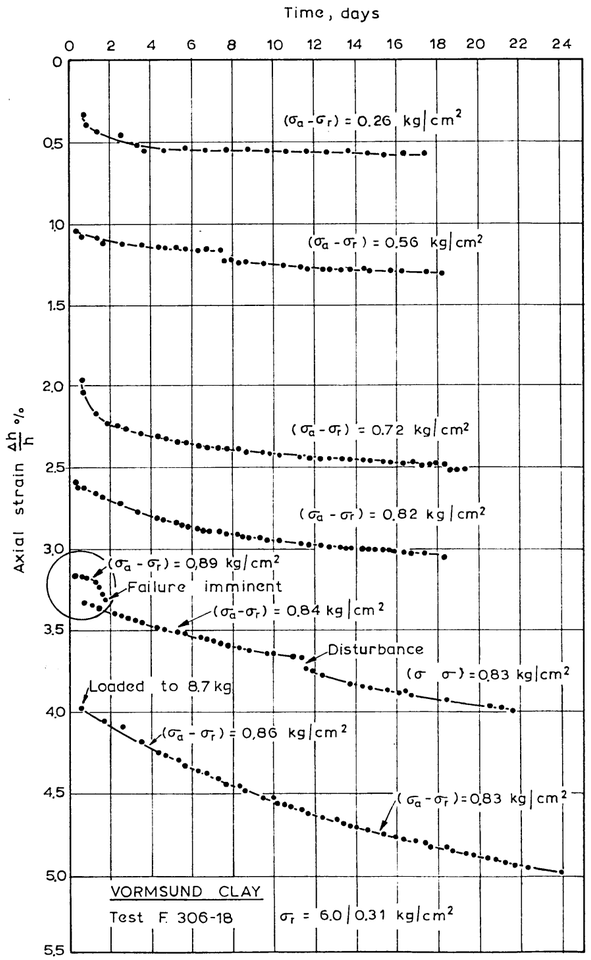


Compressibility characteristics

EFFECT OF STRAIN RATE AND TIME ON THE DRAINED SHEAR BEHAVIOUR OF A CEMENTED CLAY

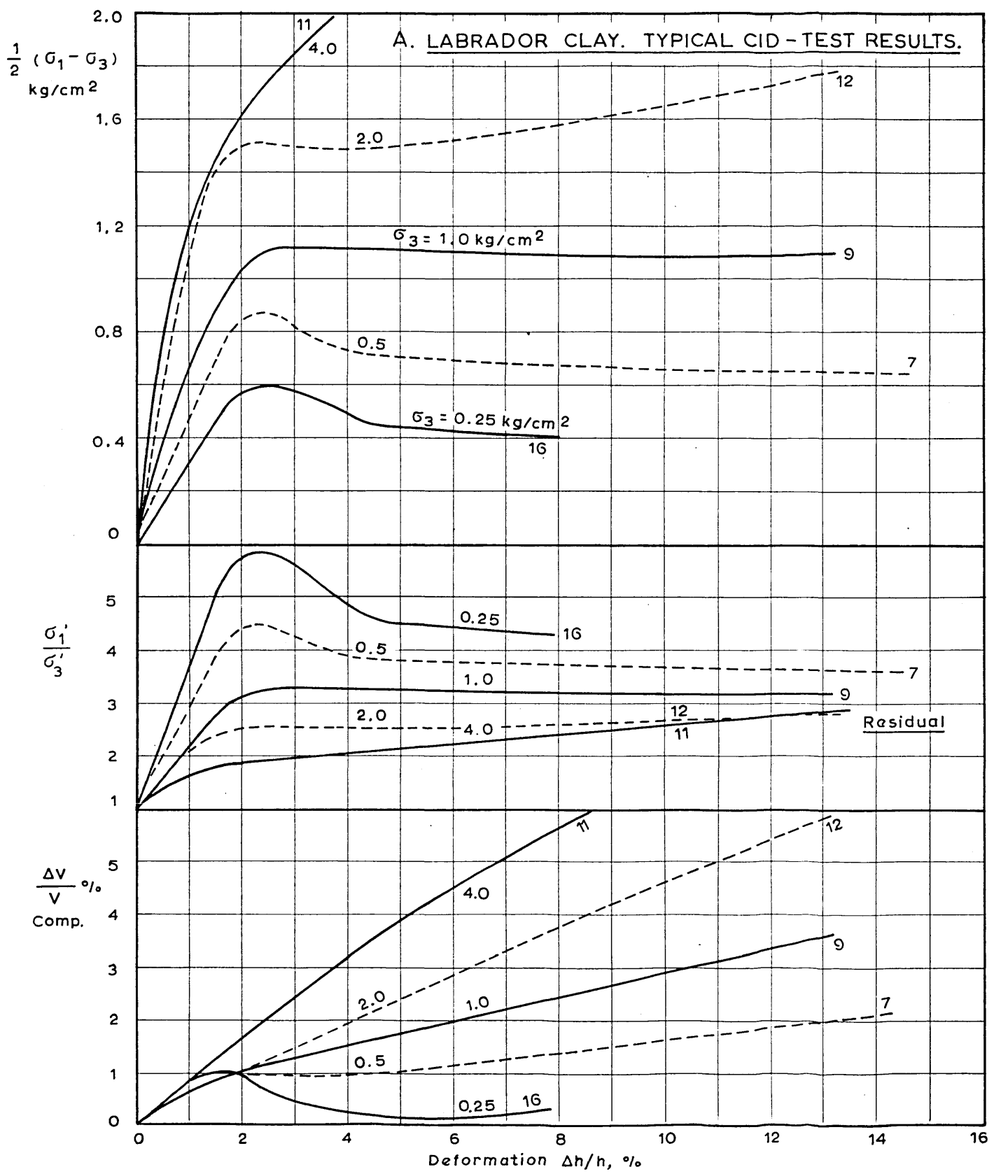


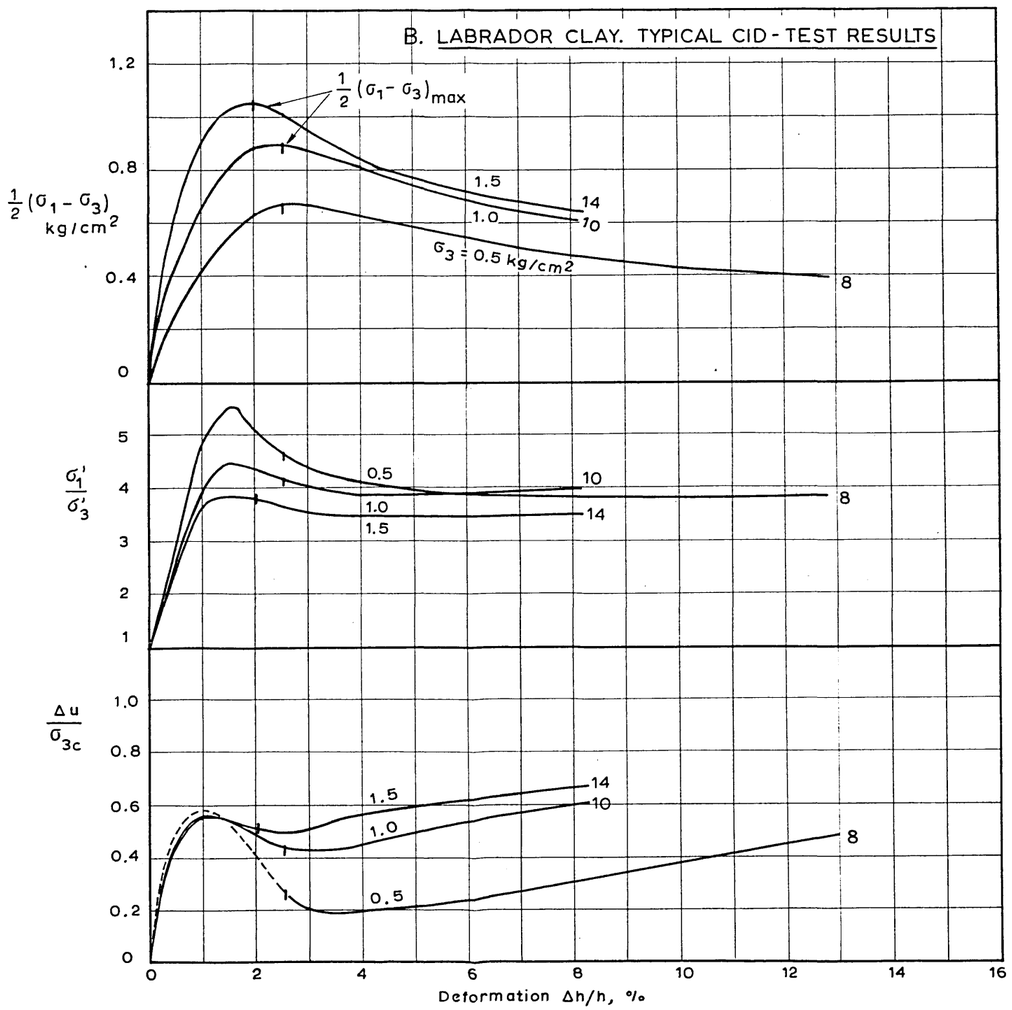


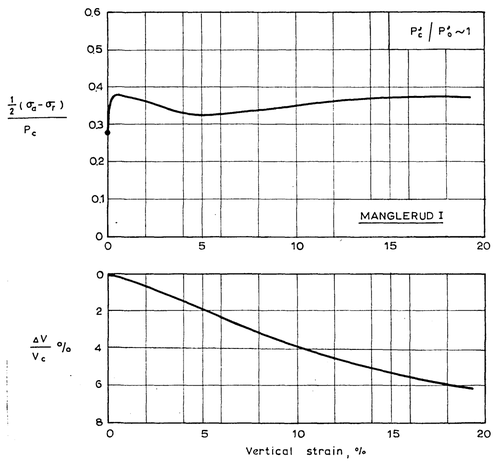


TYPICAL RESULTS OF DRAINED TRIAXIAL CREEP TESTS.

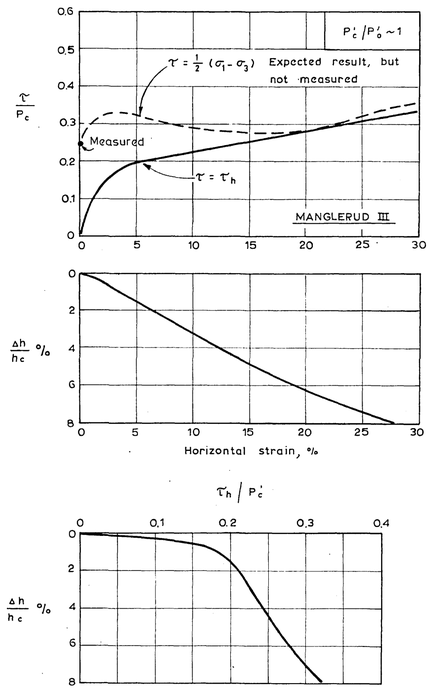
BEHAVIOUR OF A BONDED CLAY IN TRIAXIAL COMPRESSION TESTS



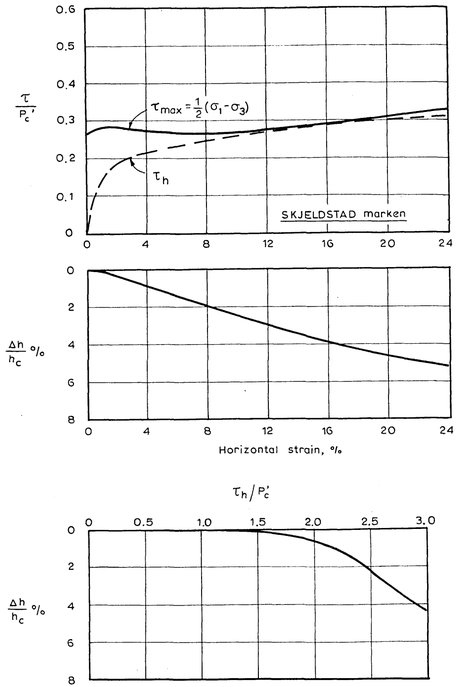




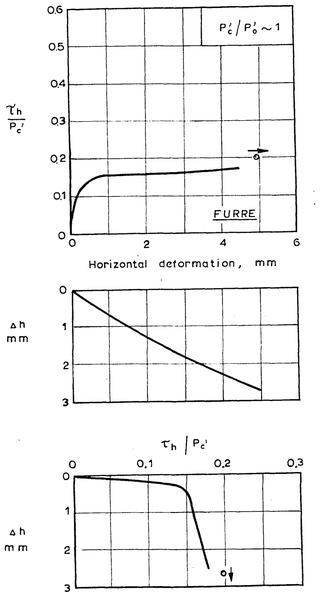
A. DRAINED TRIAXIAL COMPRESSION TEST, K_o-CONSOLIDATED
(BJERRUM AND LANDVA, 1966)



B. DRAINED DIRECT SIMPLE-SHEAR TEST, K_o-CONSOLIDATION
(BJERRUM AND LANDVA, 1966)



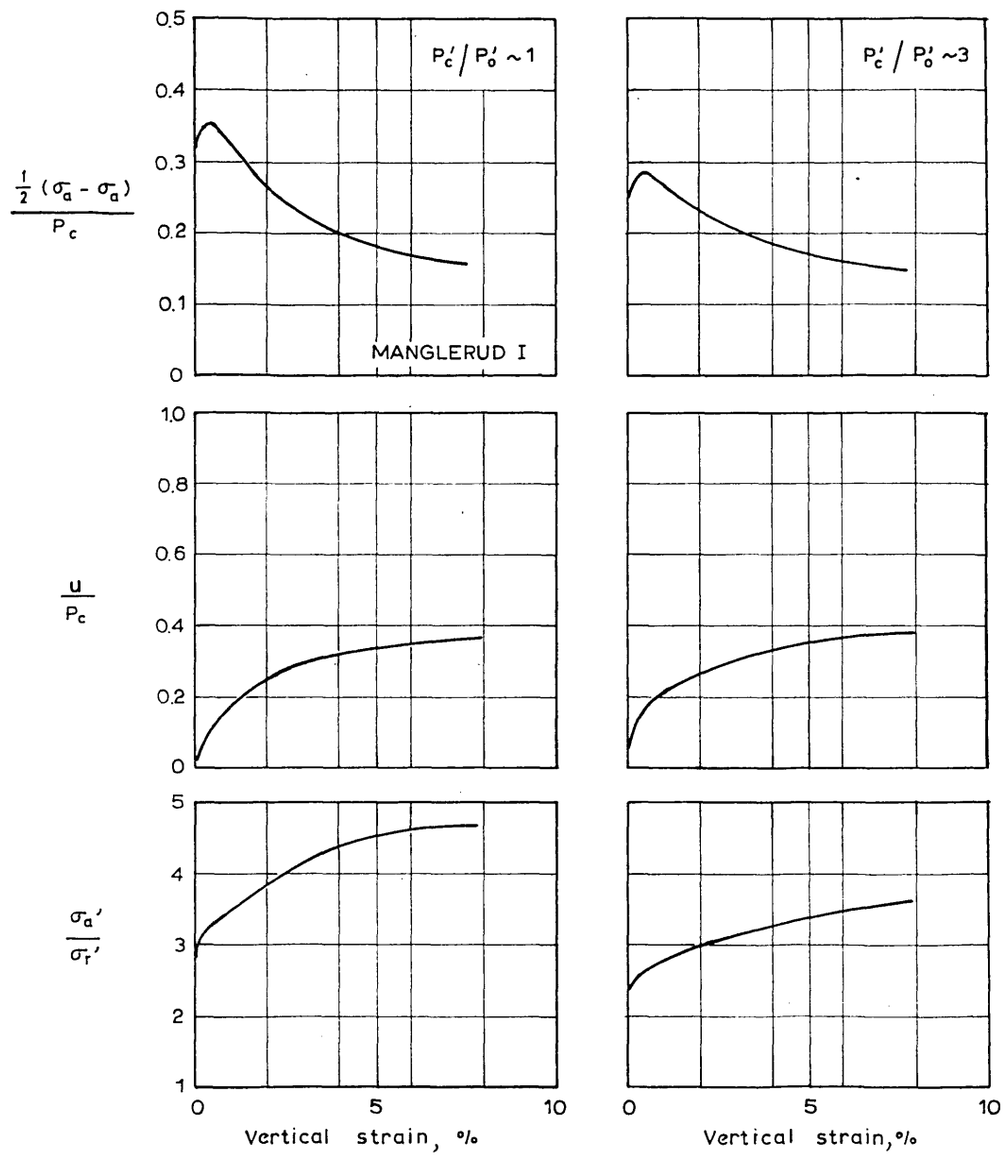
C. DRAINED DIRECT SIMPLE-SHEAR TEST
(SCHMERTMANN, NGI)



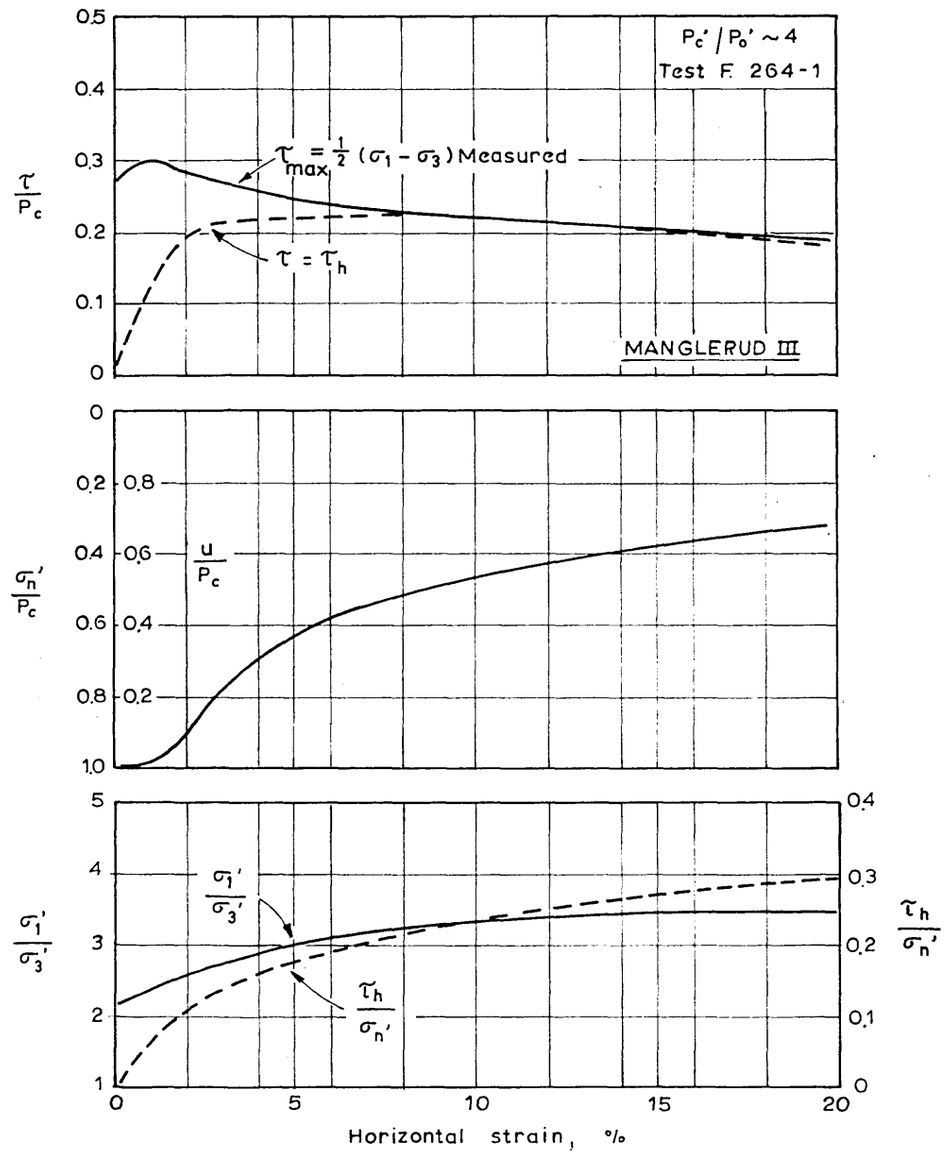
D. DRAINED FIELD SHEAR-BOX TEST
(HUTCHINSON AND ROLFSEN, 1962)

DRAINED TRIAXIAL COMPRESSION AND DIRECT TESTS ON QUICK CLAYS

Fig. 5-7

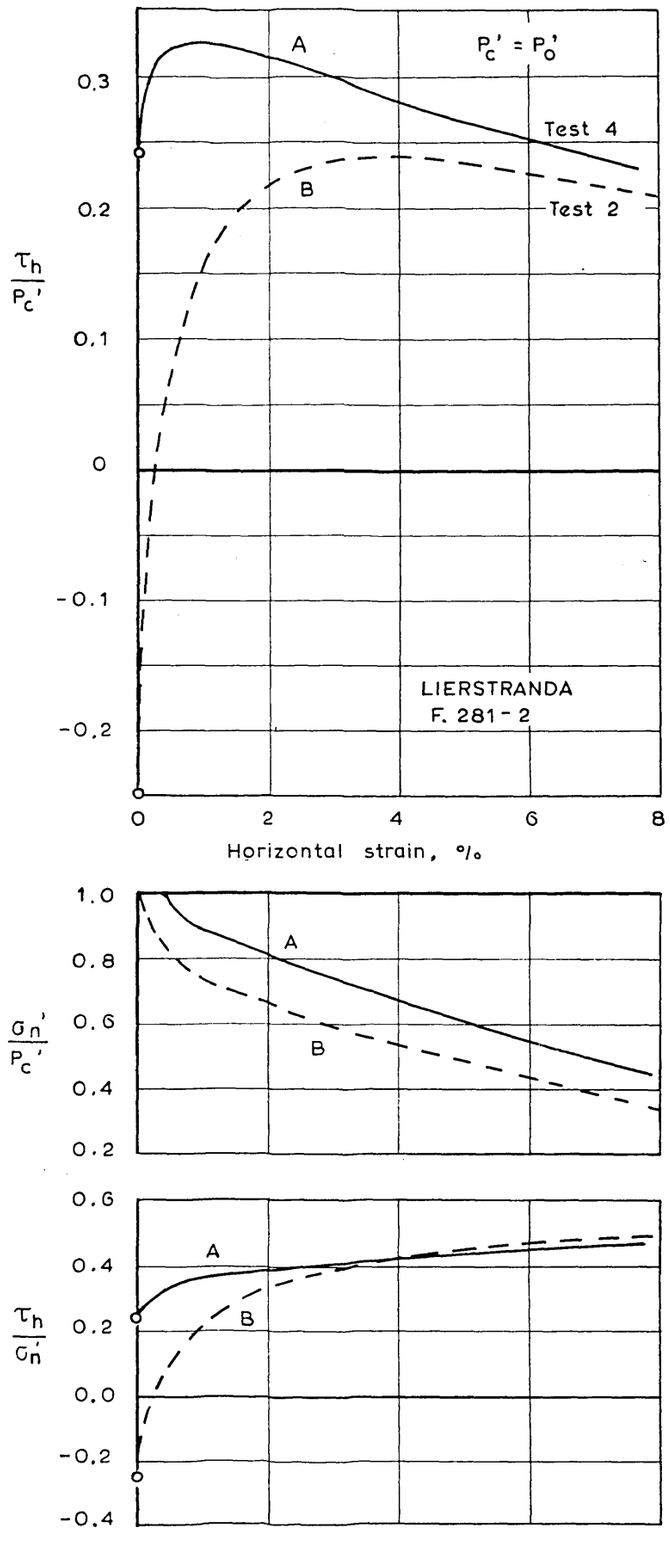


A. UNDRAINED TRIAXIAL COMPRESSION TESTS, Ko - CONSOLIDATION
(BJERRUM AND LANDVA, 1966)



B. CONSTANT VOLUME DIRECT SIMPLE - SHEAR
TEST, Ko - CONSOLIDATION

COMPARISON OF CONSTANT VOLUME TRIAXIAL COMPRESSION AND DIRECT SHEAR TESTS ON QUICK CLAYS.



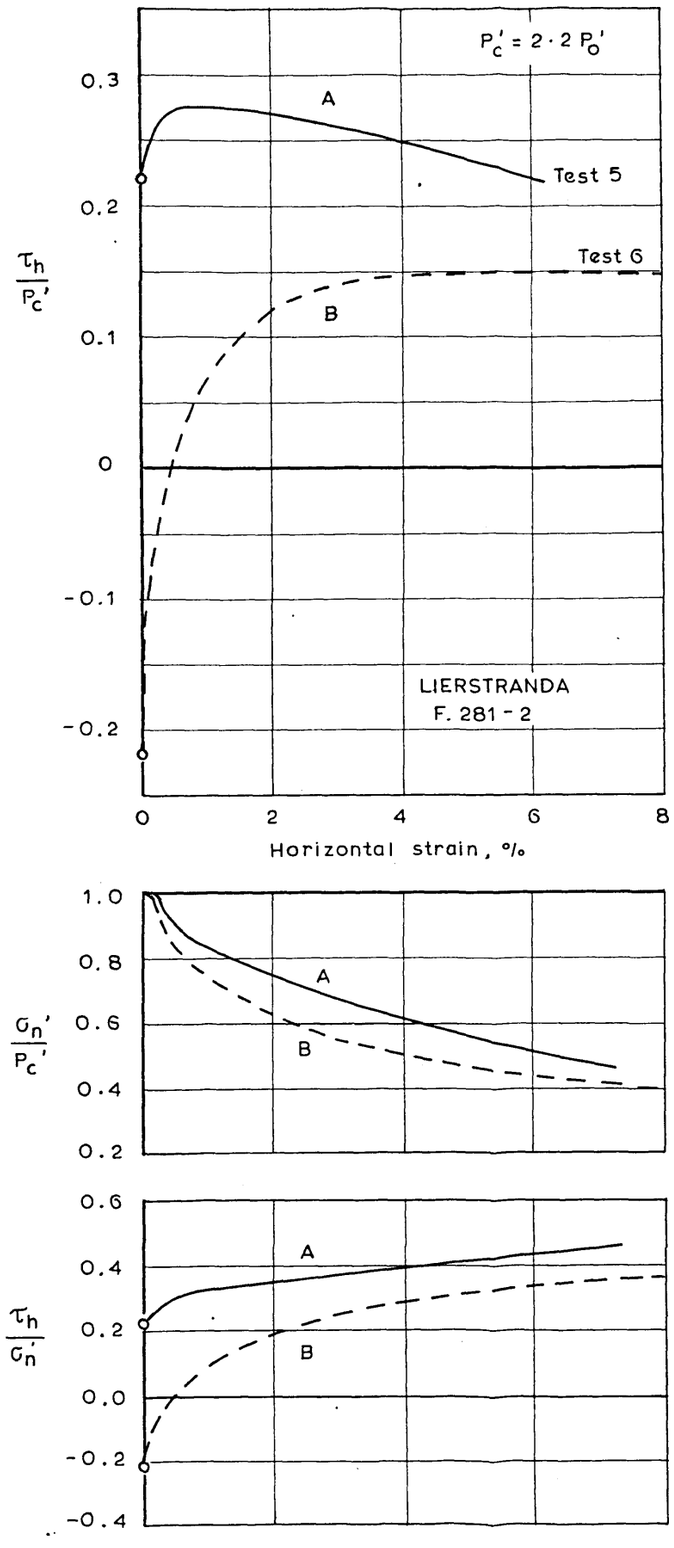
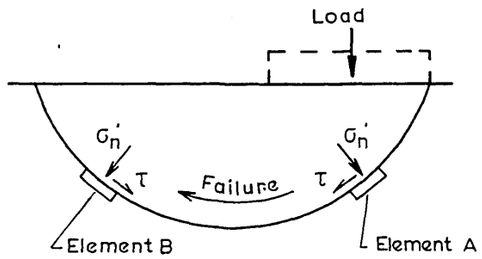
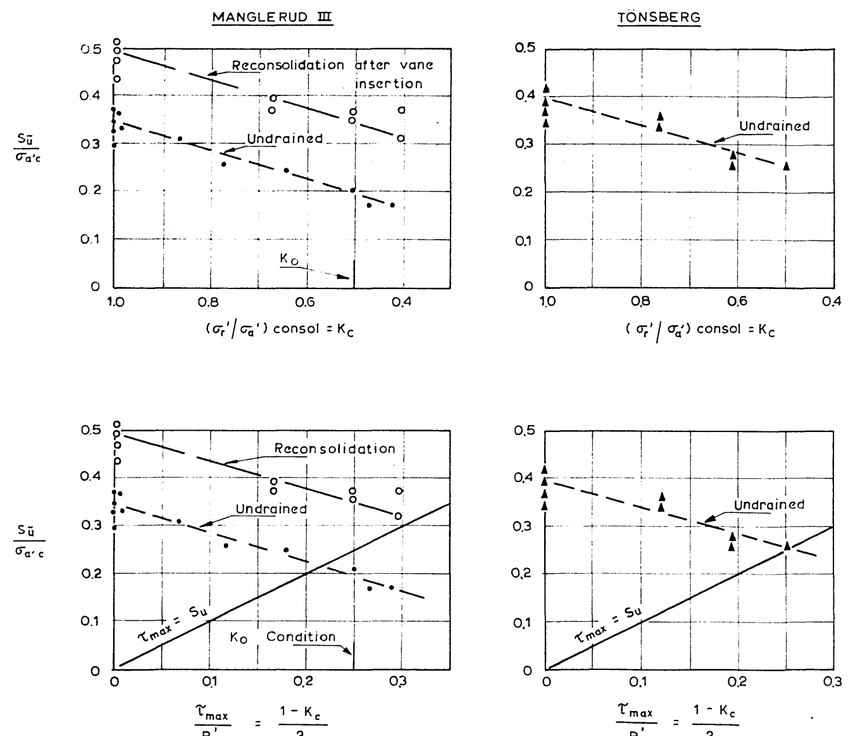


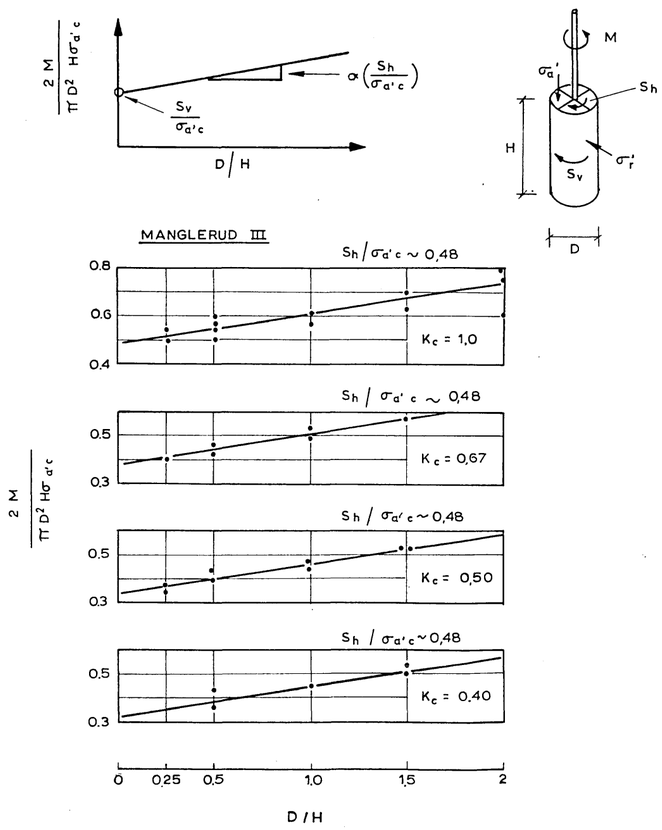
Fig. 5 - 8



DIRECT SIMPLE - SHEAR TESTS SHOWING
ANISOTROPY OF UNDRAINED SHEAR STRENGTH

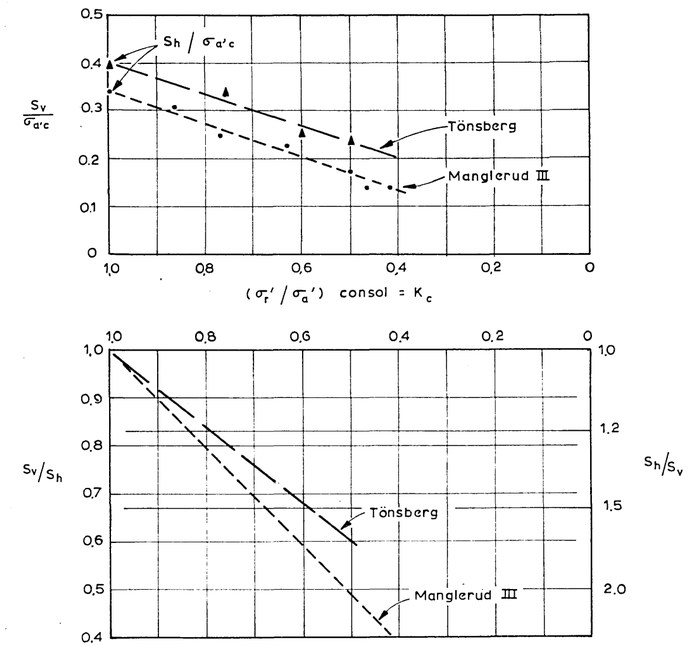


A. VARIATION OF AVERAGE VANE STRENGTH WITH RATIO OF CONSOLIDATION STRESSES (VANE D/H = 0,5)



NOTE: All samples reconsolidated after vane insertion.

B. DETERMINATION OF STRENGTH ANISOTROPY BY USING VANES OF DIFFERENT SHAPE.



C. RELATIONSHIPS BETWEEN STRENGTH ANISOTROPY AND STRESS ANISOTROPY FOR TWO CLAYS. (UNDRAINED TESTS)

RESULTS OF VANE TESTS IN SAMPLES ANISOTROPICALLY CONSOLIDATED AND TESTED IN TRIAXIAL CELLS

Acknowledgements.

To the Norwegian Geotechnical Institute, and especially to its director, Laurits Bjerrum, I wish to express my gratitude for the research opportunities and assistance I have received during my six years of employment. This entire period was one of tremendous interest and activity among the whole NGI family, and I was encouraged to follow independent lines of inquiry and was given full support at all times. My many discussions with Dr. Bjerrum were invaluable, his ideas being inspiring, his questions penetrating and his criticisms constructive. To him I am deeply indebted.

I am also indebted to a great number of people at the Institute with whom I have worked very closely. In the case of field instrumentation and measurements I wish to thank Arild Andresen, Arne Angvik, Kjell Øien, Elmo DiBiagio and Fredrick Claridge for their assistance with the K_o pipe and with the many problems that arose concerning it.

In the fields of geochemistry and mineralogy, I profited greatly from very close cooperation with Johan Moum, Olav Ihlen Sopp and Professor Robert Miller. These gentlemen accepted with great patience many questions that I now realize were ill-prepared, and it is only hoped that they gained a little from our cooperative work from which I gained so much.

I had the pleasure of working closely with Paul Drury and Egil Rolfsen on both field and office aspects of the slope-stability studies. This work was detailed and often tedious, and their constant interest and diligence are greatly valued.

In the laboratory I have been fortunate to have worked with Arvid Landva, Toralv Berre and John Schmertmann. Arvid was responsible for a large number of improvements of the apparatus used to test quick clay. Toralv must be given much of the credit for developing the K_o-membrane, and John made a great contribution to our studies of quick clay by developing a method of stress analysis for the NGI direct simple-

shear test. In addition I take great pleasure in acknowledging the careful laboratory testing work of Vivi Höyer, Jacob Rosenkrantz, Einar Stensby, Toru Shibata, Leif Ruud, Knut Holm and Fredrik Eliassen. Without their conscientious efforts the laboratory programs might easily have been failures.

The drawings have been made by Marit Sommerfeldt Herzog and Randi Wibye, and Elisabeth Topp prepared the typewritten text. Their patience and willing cooperation is greatly appreciated.

Appendix 1. Method of Preparation of Homoionic Clay-Mineral and Natural-Soil Samples.

1. Each material was mixed in a 0.5 M solution of NaCl in the weight ratio of about 1:20 to 1:40 soil to water, and then it was sedimented with the use of Sharples Super Centrifuge. This operation was repeated at least three times. These operations caused the replacement of exchangeable cations by sodium cations, and reduced the concentration of cations other than sodium in the pore fluid to negligible values.

2. The material was then mixed with distilled water and sedimented several times until the NaCl content in the pore fluid was less than 0.5×10^{-4} M. In this state the material was fully dispersed.

3. The material was separated at the required grain size by sedimentation, gravity sedimentation for minus 2μ size, and with the use of the centrifuge for smaller particle size limits. The coarse sediment was processed a total of three times to ensure that all the fines had been removed.

4. The fine fraction from this operation was used to prepare sodium, potassium and calcium samples. The salts used were NaCl, KCl, $CaCl_2$ and $CaSO_4$, and the procedure used was the same as that given under (1) except that the concentration of the salt solution could be varied depending on what concentration was required in the final sample. To reduce the salt concentration to practically zero, the soil was mixed with distilled water and sedimented a number of times, the number depending on the soil-water concentrations that were used. For samples in the Na-state, or in the Na-Calgon-state, this procedure of reducing the salt concentration was extremely lengthy because the systems were fully dispersed and sedimentation required much time.

Appendix 2. Determination of Soil Mineralogy.

Chemical tests.

Carbonates were determined by HCl treatment, and organics by wet combustion with chromic acid and determination of developed CO_2 pressure. Results were calculated as equivalent $CaCO_3$ and C, respectively.

Cation exchange capacity of a soil depends on mineralogy and hence it often can be used to indicate the presence or absence of certain minerals.

1. Sample is washed three times in a centrifuge with 0.5 M $CaCl_2$ solution, thus converting it to a calcium state.

2. Sample is washed three times with distilled water to remove excess calcium (not adsorbed).

3. Sample is washed three times with 1 M NaCl converting sample to a sodium state. Ca is determined in effluent by titration with EDTA and expressed as milliequivalents calcium adsorbed per 100 g sample (CEC).

Differential thermal analysis.

Quantitative determination of soil mineralogy by DTA performed with 5-channel simultaneously-recording Geonor apparatus using Al_2O_3 as reference material. Sample weight $\approx$ 0.8 gram. Semi-quantitative determinations of quartz (other than chert) were performed on preheated (800°C) samples by accurate determinations of the α-β quartz reaction deflections and comparing with standards of known quartz contents, procedure by Moum and Furrer (1966). Possible chert content was corrected for by X-ray diffraction analysis.

X-ray diffraction.

Qualitative and semi-quantitative determinations of mineralogy were performed by X-ray diffraction analysis. Tests were

performed on oriented samples sucked on to porous ceramic discs, employing Cu K α 1 radiation, Ni-filter. Samples were tested (1) in dry state, (2) in glycol saturated state, (3) after heating to 600°C for two-hours, and (4) after HCl treatment.

Summary of mineral identification criteria:

Mica	: by ≈ 10 Å reflection in dry state
Hydrous mica and illite	: by developments of shoulders on 10 Å low angle side in dry state
Chlorite	: by ≈ 14 Å reflection in dry and heated state
Vermiculite	: by disappearance of 14 Å line after heat treatment
Montmorillonite	: by reflections in 17 Å region after glycol expansion
Kaolin	: by persistence of 7.2 Å reflection after HCl treatment
Quartz	: by 4.26 Å reflection
Feldspars	: by reflections in 3.19-3.25≈ region.
Calcite	: by 3.03 Å reflection in dry state

Semi-quantitative determinations were performed along the lines suggested by Johns, Grim and Bradley (1954), modified by Jörgensen (1965) and Sopp (1966) utilizing the additional knowledge of the quartz contents obtained by DTA. By testing standards of known compositions and by the method of known additives, it was assumed that equal quantities of minerals would give the following relative intensities in the X-ray diffraction diagram, always making suitable corrections and being very careful when mixed layer minerals were present.

Montmorillonite glycol expanded	Chlorite Vermiculite	Mica Hydrous mica	Kaolin
17 Å	14 Å	10 Å	7.2
0.5	1	2	2

Quartz	K-Feldspar	Na-Feldspar	Calcite
4.26 Å	3.25 Å	3.19 Å	3.03 Å
1	1	3	2

Appendix 3. Disturbance of quick-clay samples from Manglerud.

1. Pore-pressure measurements as a means of detecting sample disturbance.

One method of studying disturbance and damage of samples is to measure their pore-pressure characteristics. It can be shown that undamaged samples that have not experienced volume change will be subjected to an isotropic effective stress $\bar{\sigma}'$ that is equal to or greater than the in situ radial stress $K_o P_o'$; that is, $\bar{\sigma}' \geq K_o P_o'$. In an unconfined state this stress is transferred to the soil structure by a negative residual pore pressure u_{res}, where $-u_{res} = \bar{\sigma}' > K_o P_o'$. For the special case of isotropic linear-elastic soil, $-u_{res} = 1/3(P_o' + 2K_o P_o')$. If measurements of residual pore pressure indicate that $-u_{res} < K_o P_o'$, this means that the samples are

(a) undamaged and have swelled,
(b) damaged, or
(c) both damaged and have swelled.

Some idea of the residual pore pressure in a specimen after sampling can also be obtained from the compressibility characteristics of the specimen at small confining stresses. The specimen will not experience volume changes until the pore pressure becomes positive, and therefore, if volume changes result from the application of a confining cell pressure σ_r it is known that $-u_{res} < \sigma_r$, and if volume changes do not result it is known that $-u_{res} > \sigma_r$.

The method that was used in the laboratory to measure residual pore pressures was to mount samples in triaxial cells, to apply cell pressures σ_r in increments and to measure the resulting pore pressures u for conditions of constant volume (fine-pore ceramic filters were used to prevent specimen swelling during mounting of the specimen). The residual pore pressures were equal to

$$u_{res} = -\sigma_r'$$

$$= -(\sigma_r - u)$$

Typical results from two such tests on Manglerud III clay are given in Fig. A. 3-1.

The small changes of u_{res} for cell pressures up to $\sigma_r = 1\,kg/cm^2$ have been interpreted as being test errors resulting from air in the pore-pressure measuring system that became compressed and partly dissolved at the larger cell pressures. The values taken as being correct are those drawn as full lines.

The results of the measurements of residual pore pressures and volume changes are given in Fig. A. 3-2 for samples from the sites Manglerud I and Manglerud III. Two enveloping curves for each group of tests have been drawn, the upper curve representing the behaviour of the least-disturbed samples and the lower curve representing the behaviour of the most-disturbed samples. For both sites the upper curve intersects the constant volume line ($\Delta V/V_o = 0$) at $\bar{\sigma}'/P_o' \approx 0.12$, indicating that:

$$-u_{res} \approx 0.12\,P_o'$$

$$\approx 0.24\,K_o P_o', \quad \text{for } K_o = 0.5$$

This value of residual pore pressure is much less than the minimum value for undamaged samples which is $-u_{res} = K_o P_o'$, and on this basis it is concluded that all the samples represented in Fig. A. 3-2 were disturbed by swelling, damage (plastic strains) or both.

In the following two sections, two causes of disturbance will be considered: swelling and stress change.

2. Disturbance of swelling.

When soils are samples in the field or cut in the laboratory for testing purposes, a thin zone around the sample is damaged and the soil in this zone varies from completely remoulded at the outer surface to undamaged at some distance in from the surface. This damaged zone is in the case of very sensitive clays much more compressible than the

undamaged clay, and if there existed negative residual pore pressures in the undamaged clay these would cause water to be drawn from the damaged soil (consolidation) into the undamaged soil (swelling) with the result that the effective stress in the sample would decrease from the initial value $\bar{\sigma}_o'$ to some value $\bar{\sigma}'$.

In Fig. A. 3-3 are given typical results of tests made to determine the expansibility of undamaged clay and the compressibility of the same clay in a fully remoulded state. The undamaged clay was first consolidated isotropically to a stress approximately equal to $K_o P_o'$ and then it was allowed to swell. As shown, it swelled very little and this is characteristic of these very sensitive non-plastic clays. In a completely remoulded state, which is the ultimate degree of disturbance, the soil was very compressible.

Consider the case of a very long cylindrical sample of radius r consisting of a core of undamaged soil surrounded by a zone of completely remoulded clay of thickness t, these two materials having the volume-change characteristics given in Fig. A. 3-3. Before the outer zone was damaged it is assumed that $\bar{\sigma}_o'$ within the sample was o. 25 kg/cm'. After the outer zone is damaged internal volume changes will occur for the condition of constant volume of the specimen as a whole. The change of $\bar{\sigma}'$ will depend on the thickness of the damaged zone and the radius of the sample, as expressed in the following equation:

$$\Delta \text{Vol. (swelling)} + \Delta \text{Vol (compression)} = 0$$

$$(\text{Area undamaged}) \frac{\Delta e_s}{1+e_o} + (\text{Area damaged}) \frac{\Delta e_c}{1+e_o} = 0$$

$$\pi(r-t)^2 [\Delta e_s]_{\bar{\sigma}'}^{\bar{\sigma}_o'} + 2\pi r t [\Delta e_c]_o^{\bar{\sigma}_o'} = 0$$

where $r >> t$, this expression simplifies to:

$$r[\Delta e_s]_{\bar{\sigma}'_o} + 2t[\Delta e_c]_{\bar{\sigma}'_o} = 0$$

In Fig. A. 3-3 are given the results of calculations of $\bar{\sigma}'/\bar{\sigma}'_o$, the ratio of the final effective stress to the initial effective stress, for different values of t/r. These results indicate that a completely remoulded zone equal in thickness to about 1% of the sample's radius can cause a 50% reduction of $\bar{\sigma}'_o$. The damaged clay surrounding an undamaged sample is not completely remoulded and to give comparable volume changes the thicknesses of the damaged soil would probably have to be several times greater than those shown in Fig. A. 3-3. The results thus indicate that large changes of $\bar{\sigma}'$ can result from internal swelling but that (i) great thicknesses of damaged soil or (ii) great compressibilities of the damaged soil are required to produce $\bar{\sigma}'$ -values that approach zero.

3. Disturbance by reduction of ratio of effective principal stresses.

As a consequence of sampling most samples experience stress changes, and perhaps most important are the reductions of the ratio of the principal effective stresses. If as the result of such stress changes only elastic strains occur within the sample, then the sample is disturbed but is undamaged. If, however, these stress changes cause plastic strains within the sample, then the sample is permanently damaged.

The laboratory-test method that was used to investigate disturbance by stress-ratio reduction was the same as that reported by Ladd and Lambe (1964) and Skempton and Sowa (1963). Samples were first stressed anisotropically and consolidated against a back pressure in triaxial cells. Pore pressures were then measured for the condition of constant volume of the samples while the axial stress was decreased in increments until $\sigma_a \approx \sigma_r$. the cell pressure remaining constant. The unloading period lasted from 6 to 8 days to ensure that equilibrium had been attained at each stress condition. Test results are given in Fig. A. 3-4 and are summarized in Table A. 3-1.

Of significance was the finding that after the samples had been stress relieved to an isotropic stress condition there existed in all the samples, except test 15, positive pore pressures, meaning that $\sigma_r' < \sigma_{rc}'$. Because the maximum condition for undamaged samples is $\sigma_r' = \sigma'_{r_c} = K_o P_o'$, this result indicates that these samples were damaged by reducing the stress ratio. The results were checked by consolidating a series of samples under anisotropic stresses, reducing σ_a for σ_r = constant, and measuring the volume changes for the samples. The results of these tests are summarized in Table A.3-2: for all tests, water was expelled from the samples by reducing σ_a to become equal to σ_r, and volume decreases in these tests are equivalent to pore-pressure increases in the undrained or constant volume tests.

The behaviour of the Manglerud III clay during reduction of the stress ratio at constant volume is very similar to the behaviour of normally-consolidated remoulded Dorking clay (Skempton and Sowa. 1963; Table A.3-1). For both soils positive changes of pore pressure occurred. compared with negative changes of pore pressure measured in Kawaski and Boston Blue clays (Ladd and Lambe, 1964; Table A.3-1). The only soil that behaved as an isotropically elastic material for which $\bar{\sigma}' = \bar{\sigma}_c'$, was overconsolidated remoulded Dorking clay (Table A.3-1).

The general cause of the damage resulting from decreasing σ_a/σ_r is the occurrence of plastic strains within the soil fabric that occur when recoverable strain energy is being dissipated and the sample is extending in length.

The conclusion from these tests is that, for sensitive non-plastic clays of the Manglerud type, samples are damaged (plastic strains) due to reduction of the stress ratio σ_a/σ_r. or more directly are damaged when plastic strains accompany elastic strains during stress reduction.

4. Disturbance in consolidation tests.

In this section attention will be given to disturbance (or damage) occurring in test specimens during standard oedometer consolidation tests, during specimen consolidation within reinforced rubber membranes used for direct simple-shear tests, and during K_o-triaxial consolidation tests.

It is believed that with present sampling techniques (fixed-piston, large diameter) reasonably good samples of sensitive clay are obtained. To mount specimens of these samples in oedometer, direct-shear or triaxial apparatus, the samples must be cut (trimmed) and in so doing a thin zone surrounding each sample is remoulded. This remoulded zone is very much more compressible than the undamaged clay in the case of quick clay, and therefore upon loading in the oedometer test there occur radial strains resulting from the reconsolidation of the remoulded zone (Fig. 2-1). The conditions are even worse if a space exists between the sample and the oedometer ring due to poor laboratory technique; such an initial condition probably exists when reinforced-rubber membranes of the type for direct simple-shear tests are used. These radial strains cause shear strains that cause additional damage to the sample.

Exactly the same problem exists for K_o-triaxial consolidation tests if the initial cross-sectional area of the specimen is maintained constant throughout the test. However, a major part of this source of error can be eliminated for triaxial specimens by using small isotropic consolidation pressures at the beginning of the test to reconsolidate the zone of remoulded clay. In the case of samples from Manglerud the pressures used were 0.1 to 0.2 kg/cm^2 and the resulting volume changes were from 1 to 2%, and this is a very significant amount. New sample areas were calculated and maintained constant during the following K_o tests. These triaxial K_o tests gave smaller compressibilities at pressures less than P_c than the oedometer tests and always provided more pronounced changes of compressibility in the vicinity of P_c.

This finding is clearly illustrated in Fig. A. 3-5 where the results of various consolidation tests are given for Manglerud III samples taken from a depth of about 7 m. The first number used to identify a test result is the sample tube number, and therefore triaxial K_o tests 37-2

and 37-3 were performed on material from the same tube as oedometer test 37-1. These results show quite clearly the effect of the zones of remoulded clay and indicate quite clearly that the triaxial K_o test, if carefully performed, will give a more reliable result than standard oedometer tests on very sensitive clays. These results also indicate that for strength tests, samples reconsolidated in reinforced rubber membranes for direct simple-shear testing are damaged to greater degrees than samples K_o-consolidated in triaxial cells after first re-consolidating the peripheral remoulded clay. Obviously, the problem arising from the presence of a remoulded-clay zone is one of relative thickness of the zone, and if very large oedometer or direct-shear samples were used the influence of the remoulded zone would diminish.

Table A.3-1. Results of undrained tests in which the stress ratio σ_a/σ_r was reduced.

Test no.	Consolidation history: P_c'	σ_{ac}'/P_o'	$(\sigma_a'/\sigma_r')_c$	$\bar{\sigma}_c'$	After undrained unloading: σ_a'/σ_r'	$\bar{\sigma}'$	Ratio of $\bar{\sigma}'$: $\bar{\sigma}'/\sigma_{ac}'$	$\bar{\sigma}'/\bar{\sigma}_{rc}'$	$\bar{\sigma}'/\bar{\sigma}_c'$	ϵ_u %	Remarks
Manglerud III clay (natural)											
			Units kg/cm^2								
45-07	0.47	1.30	0.611 0.342	0.432	0.346 0.322	0.330	0.53	0.97	0.76	-0.26	$I_p \approx 12\%$
43-13	0.30	6.92	2.075 0.955	1.328	1.038 0.948	0.978	0.47	1.02	0.74	-0.33	St = 30-40
56-15	0.40	1.57	0.631 0.325	0.416	0.355 0.337	0.343	0.55	1.07	0.82	-0.34	
56-17	0.42	1.27	0.534 0.320	0.391	0.321 0.299	0.306	0.57	0.96	0.78	-0.18	
60-19	0.34	1.56	0.532 0.213	0.319	0.199 0.188	0.192	0.36	0.90	0.60	-0.48	
60-20	0.34	2.10	0.712 0.343	0.466	0.332 0.318	0.323	0.45	0.94	0.69	-0.42	
Kawasaki clay (natural) from Ladd and Lambe, 1963.											
			Units kg/cm^2								
I	1.6	2.0	3.20 1.50	2.07	1.61 1.61	1.61	0.505	1.07	0.78		$I_p \approx 31\%$ St = 5-10
	2.6	2.1	5.40 2.50	3.47	3.30 3.30	3.30	0.610	1.32	0.95		$I_p \approx 36\%$
	3.5	2.2	7.55 3.50	4.85	3.97 3.97	3.97	0.525	1.13	0.82		$I_p \approx 43\%$
Boston Blue clay (remoulded, NC) from Ladd and Lambe, 1963.											
			Units kg/cm^2								
			6.05 3.30	4.22	3.65 3.65	3.65	0.60	1.11	0.87		$I_p = 14\%$
			3.10 1.65	2.13	1.90 1.90	1.90	0.61	1.15	0.89		
Dorking clay (remoulded, NC) from Skempton and Sowa, 1962.											
			Units psi								
28			28.0 18.0	21.3	17.0 17.0	17.0	0.61	0.94	0.80		
63			90.1 53.5	65.7	51.4 51.4	51.4	0.57	0.96	0.78	-1.4	$I_p = 24\%$
99			92.2 53.5	66.4	52.6 52.6	52.6	0.57	0.98	0.79	-0.8	
89			127.2 75.0	92.5	73.2 73.2	73.2	0.58	0.98	0.79	-0.9	
86			166.8 100.0	122.3	98.1 98.1	98.1	0.59	0.98	0.80	-0.9	
Dorking clay (remoulded, OC at P' = 170, K_oP' = 100).											
			Units psi								
79			13.5 17.0	15.8	15.9 15.9	15.9	1.18	0.94	0.99		
91			78.8 63.0	68.2	68.0 68.0	68.0	0.86	1.08	1.00	0.3	

$\bar{\sigma}' = 1/3\,(\sigma_a' + 2\sigma_r')$; σ_a' axial, σ_r' radial.

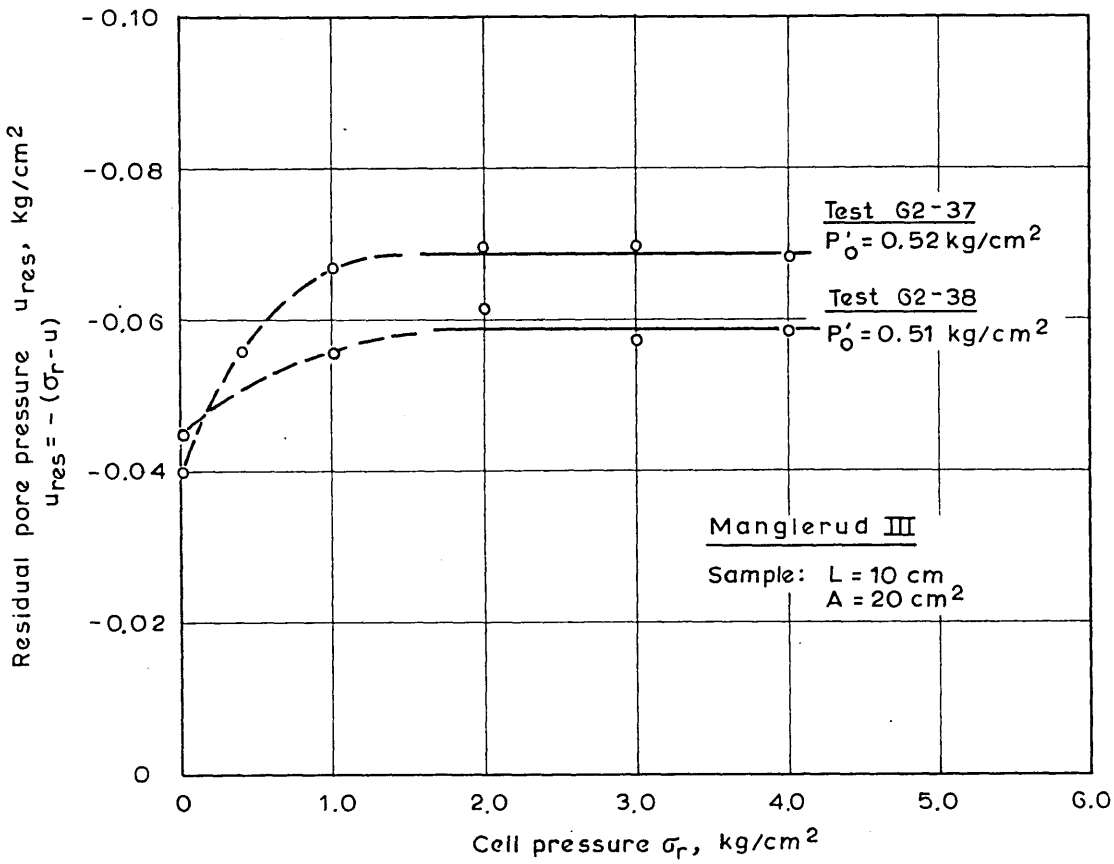
ϵ_u strain by unloading, negative values for length increases.

Table A.3-2. Results of drained triaxial tests in which the stress ratio σ_a/σ_r was reduced for σ_r = constant.

	Consolidation history			After stress reduction		
Test no.	P_o'	$\frac{\sigma_{ac}'}{P_o'}$	$(\frac{\sigma_a'}{\sigma_r'})_c$	$\frac{\sigma_a'}{\sigma_r'}$	$\frac{\Delta V}{V_o}$ %	ϵ_u %
	Units kg/cm^2					
74-25	0.41	1.51	$\frac{0.62}{0.50}$	$\frac{0.51}{0.50}$	0.35	-0.16
74-26	0.41	1.53	$\frac{0.63}{0.32}$	$\frac{0.33}{0.32}$	0.28	-0.67
68-23 A	0.28	2.96	$\frac{0.83}{0.40}$	$\frac{0.42}{0.40}$	0.20	-0.69
68-23 B	0.28	5.32	$\frac{1.49}{0.65}$	$\frac{0.67}{0.65}$	0.50	-0.84
68-24 B	0.27	5.51	$\frac{1.49}{0.68}$	$\frac{0.69}{0.68}$	0.50	-0.67

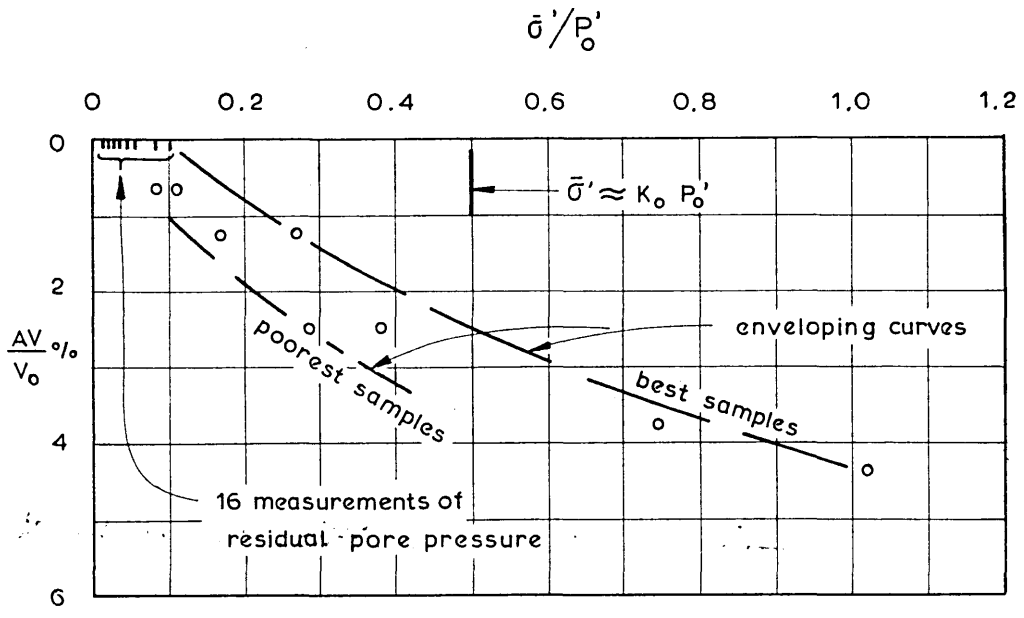
ΔV denotes volume change by unloading, positive values for volume decreases.

ϵ_u denotes strain by unloading, negative values for length increases.

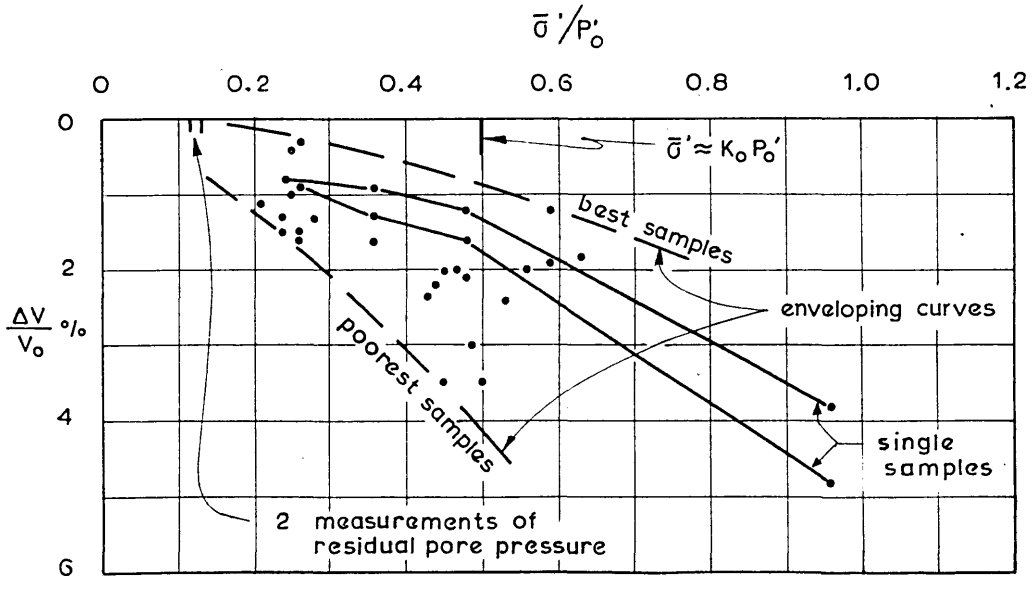


Note: Sample G2-37 built in on sampling day.
Sample G2-38 built in on 10 days after field sampling

RESIDUAL EFFECTIVE STRESS AFTER SAMPLING AND LABORATORY TRIMMING

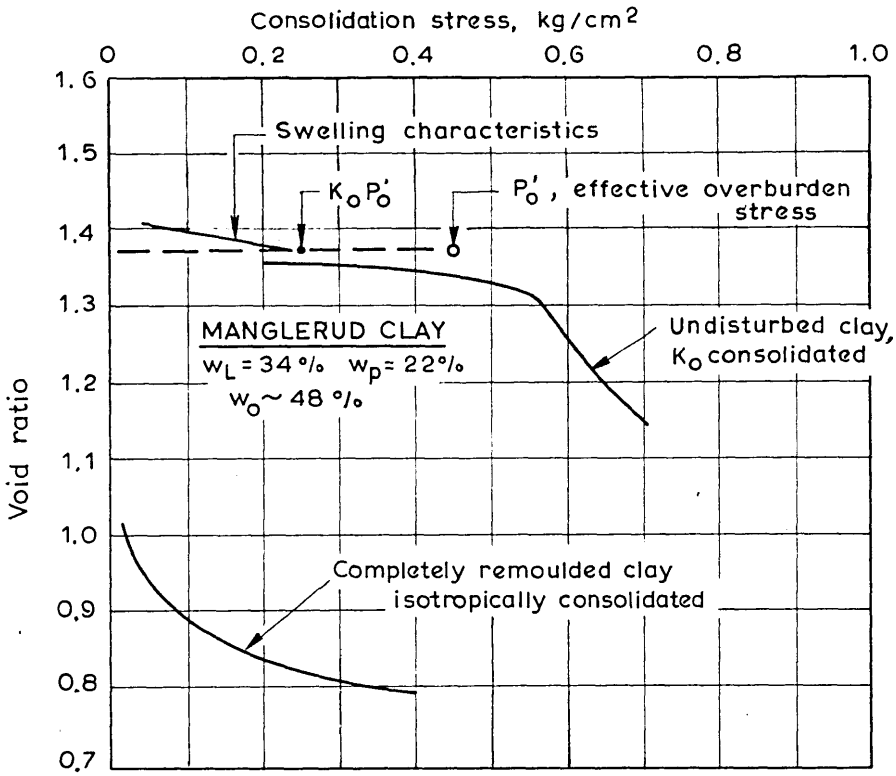


Manglerud I (F.234, tests 1-16)

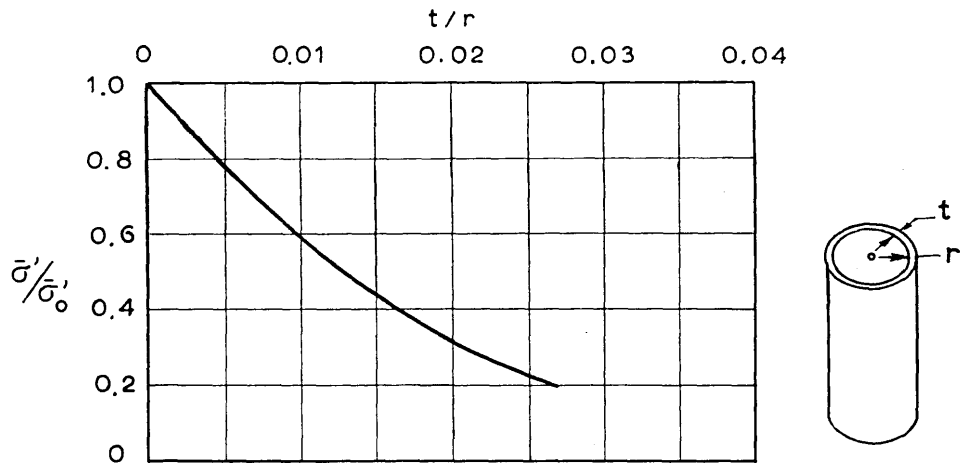


Manglerud III (F.264, tests 1-38)

MEASUREMENTS OF RESIDUAL PORE PRESSURE AND VOLUME CHANGE FOR RECONSOLIDATION

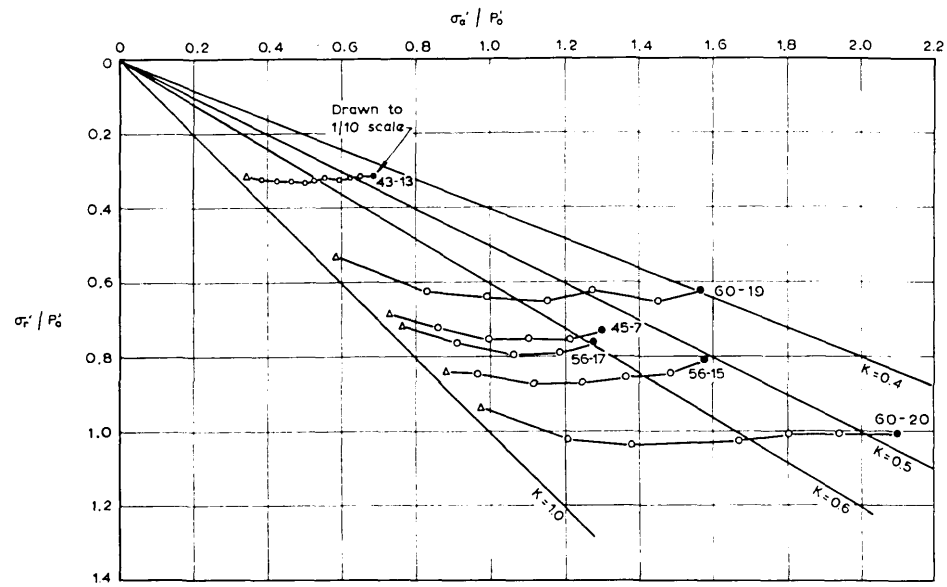


Consolidation characteristics of typical Quick Clay

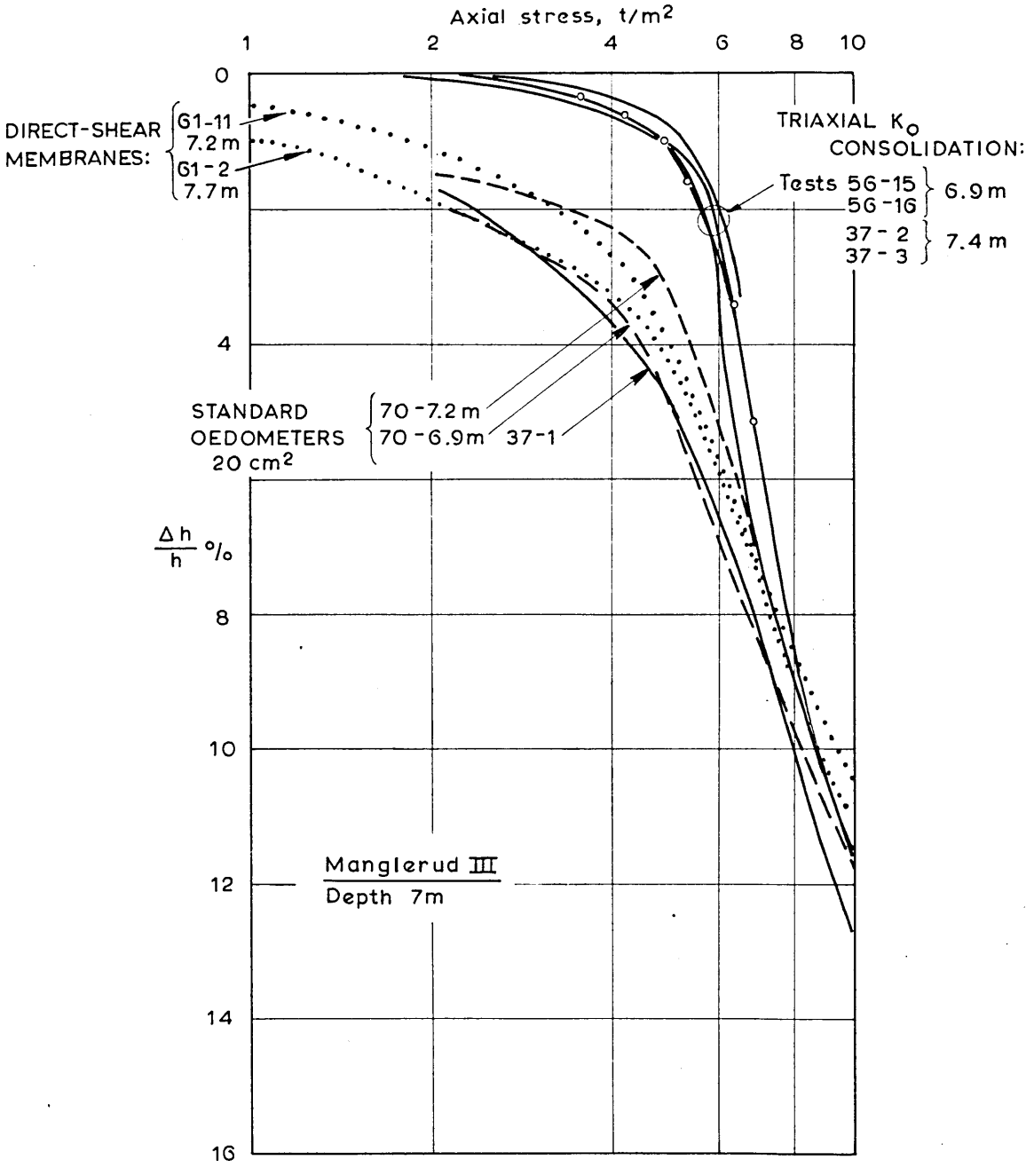


Effect of internal swelling on $\bar{\sigma}'$

EFFECT OF SURFACE DISTURBANCE OF QUICK-CLAY SAMPLES ON EFFECTIVE STRESS FOR CONSTANT-VOLUME CONDITIONS



UNDRAINED TESTS IN WHICH THE STRESS RATIO σ_a/σ_r WAS REDUCED



RESULTS OF DIFFERENT TYPES OF CONSOLIDATION TEST

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